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THE UNIVERSITY OF ALBERTA

THE UNIVERSITY OF ALBERTA PRESSUREMETER

by

NEIL CHARLES BURGESS

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A THESIS

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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled THE UNIVERSITY OF ALBERTA PRESSUREMETER submitted by Neil Charles Burgess in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

ABSTRACT

The overconsolidated soils and soft rocks which form a large proportion of foundation material in Western Canada typically have a substantial component of their total deformation of an immediate (time-independent) nature. This makes them particularly suitable for the application of pressuremeter testing when deformation characteristics are sought for settlement analyses.

This thesis describes first the development of a new pressuremeter probe which eliminates some of the shortcomings associated with the older, more conventional type. The instrument has been specifically designed for use in materials such as gravel, glacial till, lacustrine sediments and soft bedrock.

Secondly, it describes in detail pressuremeter testing on three sites in Western Canada where well documented case histories of deformation behaviour were available.

The first site was Blackstrap Mountain near Saskatoon, Saskatchewan which is an artificially filled ski hill founded on very deep till strata. The second case history was a pile load test at the new Calgary Airport where the pile was embedded in Cretaceous bedrock. The third site was one of a grain elevator, also in Calgary, placed on a layer of dense gravel.

The pressuremeter derived moduli were used in back-analysis of settlement and the results were compared with actually measured field behaviour.

Good agreement between the back-analysis and field data in some cases and discrepancies in others indicated the limitation of pressuremeter testing in this type of foundation material. The results are discussed and directions for improvement of this testing technique are proposed, if the present limits of confidence are to be extended.

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CHAPTER I

INTRODUCTION

1.1 Scope of The Thesis

The pressuremeter is an instrument with which an insitu deformation or strength test can be performed in a soil or rock mass. It is particularly suited for the foundation conditions in Western Canada where the immediate portion of the deformation of foundation soils, in many cases, is a significant part of the total deformation. In addition, the relatively high safety factors used in most foundation engineering make a linear-elastic approach to design possible.

Most of Western Canada is overlain with glacial deposits, the predominant type being glacial till of either active or stagnant ice in origin. The glacial deposits overlie relatively soft bedrock of Cretaceous age over much of Western Canada. Both the glacial till and the soft bedrock are, in general, difficult to sample with a view to obtaining deformation parameters that can be used with confidence in the prediction of foundation performance.

In the past, foundation loading has been determined largely on the basis of strength parameters with safety factors of 2 or 3 employed to limit imposed shear stresses to roughly 1/4 to 1/6 of peak deviator stresses

and thereby control foundation deformation. This procedure cannot produce a quantitative picture of deformation and is known to result often in conservative design loads.

An alternative approach to foundation design concerns the direct prediction of foundation deformation using insitu deformation parameters and the powerful finite element method of analysis. A linear-elastic analysis is then justified in most foundation engineering in the overconsolidated soils and rock that are so prevalent in this region. The stress-strain characteristics of these materials combined with the relatively high safety factors commonly used, restrict in most cases foundation deformation to the linear-elastic region.

This approach requires the determination of a representative deformation parameter for use in the above method of analysis.

The deformation modulus is known to be highly sensitive to sample disturbance which cannot be avoided in obtaining conventional laboratory specimens. The insitu measurement of this parameter using the pressuremeter is a means of overcoming the problem of sample disturbance. (Morrison, 1972).

The subject of this thesis is the development of a new pressuremeter and its application in field studies. Restrictions inherent in the design of commercially available NX probes (3 inch diameter) required the con-

struction of a more versatile probe for use in materials other than soft bedrock, particularly glacial till, lake sediments and granular soils.

The confidence with which the pressuremeter-determined deformation parameters can be applied to the prediction of the behaviour of soft bedrock, glacial till and gravel is considered fundamental to the study. The case-history approach is pursued here with the above in mind. Emphasis is placed on a pragmatic approach to the relevance of the results of the pressuremeter test. An attempt is made to accurately obtain a deformation modulus, using the pressuremeter, that can be used with confidence in a linear-elastic, two dimension plane strain or axisymmetric finite element program.

1.2 Theory Relevant to the Pressuremeter Test

The pressuremeter test is an example of the expansion of a thick-walled cylindrical cavity. Early solutions to this problem can be found in the work of Lamé (C.1830). Modifications to the theory have been made to account for fundamental differences in the behaviour of soils and metals. The derivation of material properties from measured cavity expansion is the inverse problem of that studied in metal plasticity. This derivation became possible with the construction of instruments such as the pressuremeter.

Earlier interpretations of the pressuremeter test

were based on an elasto-plastic stress-strain relationship for soils (Gibson and Anderson, 1961). Ladanyi (1961, 1963a, 1963b) has presented solutions to the problem of expanding cavities that require no presumption of stress-strain behaviour and can include the effect of volumetric straining around the cavity. Ladanyi's earlier work involved cavity expansion in a clay medium (1963a). He derived the state of stress and strain in the soil mass affected by cavity expansion. The solution for stresses involved the integration of the equations of equilibrium for the case of radial plain strain using a numerical procedure. Shear strain (distortion) was defined by geometric conditions only.

1.3 Soil Properties from the Pressuremeter Test

The pressure-volume change relationship obtained from the pressuremeter test can be used to define a number of soil properties. These include the undrained stress-strain characteristics, the deformation modulus in loading and unloading, the effective angle of shearing resistance and creep parameters in the use of frozen soils.

Ladanyi (1972) has shown for sensitive clays that the undrained shear strength obtained from the pressuremeter compares well with the results of deep cone-penetration tests and exceeds strengths obtained in the laboratory. The increased strengths are attributed to decreased sample disturbance, contained plastic flow

which diminished the influence of inherent planes of weakness and to a loading rate that exceeds that used in the laboratory. The pressuremeter test can furnish post-peak stress-strain behaviour which is difficult to establish in the laboratory, particularly in the case of sensitive, strain softening clays.

Ladanyi (1963b) and Vesić (1972) have published analyses from which the angle of shearing resistance for granular soils can be determined from the pressuremeter test. Both methods are iterative or trial procedures and include the effect of volumetric straining within the stressed soil mass. Vesić shows that the consideration of volume change within the zone of plastic deformation around a stressed cavity is important for the determination of a strength envelope. The modulus of deformation is shown to be insensitive to this consideration. Vesić's analysis is a general one for both spherical and cylindrical cavity expansion and employs tabulated parameters. Ladanyi's analysis is not so general and is considered somewhat more cumbersome.

Baguelin et al (1972) report on pressuremeter tests in cohesive soils. Their study was made with a self-placing pressuremeter, a technique used to minimize soil remoulding at the bore hole wall. Undrained shear strengths obtained with their pressuremeter are shown to exceed strengths from vane tests by roughly 50%. The determination of shear strength in soft clay

is shown to be very sensitive to remoulding at the bore hole wall due to disturbances that must occur in the course of conventional test hole drilling.

Palmer (1972) has derived a relatively simple interpretation of the pressuremeter test that is used in the derivation of the undrained stress-strain characteristics of a soil. His analysis shows the principal stress difference to be twice the gradient of a pressure-log volumetric strain relationship at any stage of the pressuremeter test, the same relationship derived by Ladanyi (1972). The methods differ with regard to the definition of shear strain around an expanding cavity.

The modulus of deformation can be derived on the basis of the analysis of Gibson and Anderson (1961). The solution to the problem of the expansion of an infinitely thick-walled cavity contains the same formula. A simplified version of this formula was employed by Morrison (1972) and is used in this study. The simplification involves the exclusion of second power strain terms and results in insignificant error for the range of volumetric straining encountered in the pressuremeter test.

The modulus of deformation can be obtained in loading and unloading in the pressuremeter test, the ratio of the moduli being related to rock quality for the case of fractured rock (Dixon, 1970).

Ladanyi and Johnston (1973) have derived creep

parameters from pressuremeter tests in frozen fine-grained sediments found in northern Manitoba. The tests were of rather limited duration with regard to the definition of these parameters, a consequence of the capacity for volume change of their NX probe. This and other limitations of commercially available probes are discussed in Section 2.1

A comparison of strength and deformation properties obtained from the pressuremeter test with those obtained by other means has been made in few reported cases.

Deformation parameters obtained in soft bedrock in the Edmonton area have been shown to relate well to building settlement and to heave experienced in excavation in these materials (Eisenstein and Morrison, 1973).

Empirical and semi-empirical relationships have been derived by Menard (1965) and others, Mori and Tajima (1965) that relate various aspects of the pressuremeter test to the performance of deep and shallow foundations. The relationships reflect an attempt to quantify the similarities between the expansion of a pressuremeter and the performance of a loaded foundation.

A more rational approach would appear to be the application of strength or deformation parameters from the pressuremeter test to proven conventional analyses.

CHAPTER II

DEVELOPMENT OF THE U OF A PROBE

2.1 Limitations of the NX Probe

A number of limitations inherent in the design of the 3 inch diameter (NX) probe, which is available on a commercial basis, have led to difficulties in its field use (Morrison 1972). The probe used by Morrison has a limited capacity for volume change. For this reason the instrument could not be used in soft soils or in test holes in which a close tolerance could not be obtained between the exterior of the probe and the test-hole wall. This limitation required the use of non-standard drilling equipment and extra care during drilling. The above NX probe was not equipped with a device that permitted its operation in a dry test hole. When the probe is lowered down a test hole a piezometric head is created on the measure cell and is equal to the difference in elevation of the water supply reservoir and the measure cell of the probe. This head can be countered by an all-round pressure applied to the exterior of the measure cell. This pressure can only be supplied to the guard cells by fluid in the test hole, such as drilling mud.

The commercial model used by Morrison had a rather complex and cumbersome control system. Considerable

effort and experience on the part of the operator was required in the operation of the probe.

2.2 Modifications to the NX Probe Design

To overcome the shortcomings of the probe described above and by Morrison (1972) a new instrument was constructed at the University of Alberta and is named the U of A probe. This instrument was larger than the NX probe, having an overall outside diameter of 4 3/4 inches and a length of 60 inches (See Figure D1).

Incorporated in the design of the U of A probe were features that resulted in a more versatile instrument. These included:

1) a pressure relief valve on the probe,
2) a more flexible measure cell with a satisfactory capacity for volume change,
3) a simplified control system.

To permit the use of the probe in a dry test hole, a pressure relief valve was constructed on the new probe. The relief valve is adjustable and can be set to open at a predetermined pressure. During lowering of the probe, the piezometric head in the water supply line to the measure cell acts on the pressure relief valve and not on the measure cell until the valve opens at a desired pressure. In this manner, drilling fluid is not required to counter the piezometric head. The

water supply valve on the control panel can be left open to avoid the occurrence of cavitation in the water supply line.

Measure cells constructed from 1/16 inch cured gum rubber sheet were found to be sufficiently flexible to permit essentially cylindrical expansion to large volumetric strains. In addition, their contribution to probe inertia was found insignificant. An air-water pressure differential of 10 psi was found sufficient to permit expansion of the measure cell at rates in excess of that encountered during testing in even the softest tills. This pressure difference had to be minimized to avoid stress concentrations at the ends of the measure cell portion of the outer membrane.

During expansion of the probe, it was essential that the measure cell portion of the probe was able to expand in a radial direction only. The measure cell portion of the probe consisted of the middle third of the expandable part of the probe. During expansion of the probe in rigid tubing in the laboratory, the external membrane was seen to be in full contact with the restraining tubing within several inches of the ends of the expandable portion of the probe. For this reason, the condition of radial plane strain was met over the measure cell portion of the probe.

2.3 Preliminary Laboratory Trials

The key feature of the U of A probe was the degree

to which expansion of the measure cell approximated to that of cylindrical expansion and at what flow rates this would occur. Theoretical analyses of the expansion of a cylindrical cavity assume that a state of radial plane strain is obtained. Measurement of volume change that is consistent with such expansion required that the probe's measure cell undergo similar or identical expansion. Volume change of the measure cell was sought such that its expansion closely paralleled that of a right circular cylinder. Experimentation with various membrane designs, pressure differentials and coaxial tubing sizes resulted in a satisfactory design. As mentioned above, measure cells constructed from 1/16 inch cured gum rubber were found most suitable. These membranes were installed such that a 15 inch length of measure cell was fitted to the middle 13 inch section of the probe, as depicted in Figure D1. The measure cell was expanded in several pipes with diameters of 4, 5 and 6 inches. Volume change was measured for various pressures on the cell and flow rates were recorded.

A plot of volume change versus the square of measure cell diameter is shown on Figure 1. From this graph, a pressure of 9 - 10 psi is seen to produce essentially cylindrical expansion of the measure cell.

A pressure of 9 - 10 psi on the water reservoir was sufficient to produce a flow rate at the probe of 500 cc/minute. This rate required a water line of 3/16

inch O.D. thin-walled thermoplastic tubing. As indicated in Figure 2, a flow rate of 500 cc/minute proved satisfactory for the soft till at the nine foot depth at Mt. Blackstrap. In the case of harder materials, the above flow rate contributed to an elastic time increment of as little as five seconds. (See Figure B1).

Volume change of the control system was measured and considered insignificant at pressures up to 200 psi.

Probe inertia was determined for a new, 3-ply outer membrane. The results are shown on Figure 3. The pressure required to inflate the probe under zero outside resistance is required only for shear strength calculations, in that applied pressure at the bore hole wall is required. Over the pseudo-elastic portion of the pressure-volume curve, the change in volumetric strain is of the order of a few percent. The pressure required to inflate the probe itself (inertia) can be considered constant over the small volume change undergone in deformation tests. For this reason probe inertia need not be considered in deformation tests.

Tests were conducted to determine whether the pressure relief valve design was adequate. A dry test hole situation was simulated by elevating the reservoir and control system to as high as 60 feet above the probe. With the probe and reservoir situated at ground elevation, the probe was inflated in a length of PVC pipe and the cracking pressure of the pressure relief

valve determined. The valve was adjusted until a desired cracking pressure was attained. In practice, this pressure would be equal to 10 psi less than the piezometric pressure on the measure cell when the probe had been lowered to a desired test elevation. With the cracking pressure determined the probe was deflated and the reservoir and control system were elevated. The probe was then inflated in the PVC housing and water flow was allowed at this point, to determine whether the desired pressure (the cracking pressure) had been attained. Several pressure increments were applied to the probe to determine whether the air-water pressure differential remained constant.

The cracking pressure determined with the probe and reservoir at the same elevation could not be attained upon deflation and elevation of the reservoir. An attempt was made to determine the required cracking pressure on the basis of flow rates to the probe. This method proved unsatisfactory.

The incorporation of a pressure relief valve on the probe to facilitate testing in dry test holes was attractive in view of soil conditions in Western Canada. Such a valve could be used in borings put down in formations within which the circulation of drilling fluid cannot be maintained. In such cases, only the groundwater table can be relied upon for the provision of an all-round pressure on the probe to counter the piezo-

metric head that develops on lowering of the deflated probe. Problems in avoiding cavitation in the water line to the probe become acute if the groundwater level is at considerable depth. Although it was not tried in the course of this work, a viable alternative to the relief valve would be an electric solenoid valve. Such a valve could be kept closed during lowering of the probe until the required depth was achieved at which time the probe could be inflated, the piezometric head countered and the electric valve then opened. Such a valve would require extensive waterproofing of the probe housing for submergence but could easily be incorporated for testing in completely dry borings.

CHAPTER III

FIELD STUDIES

3.1 Field Studies - Mount Blackstrap

3.1.1 Site Description

Mount Blackstrap was constructed southeast of Saskatoon, Saskatchewan for the 1971 Canadian Winter Games. The artificial mountain is essentially conical in shape having a base diameter of 600 feet and a height of 130 feet (Hamilton and Tao, 1972).

The underlying strata consist of two distinct till sheets overlying the Bearpaw Formation. The tills are partially saturated and are differentiated on the basis of dolomite content and weathering zones. In addition, the upper sheet is divided into two formations.

Excellent settlement data were obtained by means of a fluid settlement gauge placed diametrically across the base of the fill. In addition pore pressures and slope movements were monitored during construction. Extensive laboratory tests were conducted on the three till formations for the purpose of defining the compression characteristics of the foundation strata. The results are shown on Figure 7, from Hamilton and Tao (1972).

The site was considered suitable for a pressuremeter field study in that a high percentage of total settlement was considered immediate and substantial data existed on soil properties and foundation settlement. The case provided a means for comparison of deformation moduli obtained from the pressuremeter and laboratory tests.

3.1.2 Field Test Program

A sampling and testing program was undertaken in October, 1973 in co-operation with the Division of Building Research, NRC, Saskatoon. The drilling was provided by the Prairie Farm Rehabilitation Administration, Saskatoon.

Deformation moduli were obtained to the 90 foot depth only.

Adverse weather conditions arriving unexpectedly early hindered drilling operations and necessitated experimentation with various fluids for volume change measurement with the pressuremeter. Consequently, tests were not conducted to a desired depth.

Drilling was done with a Failing 1500 rig using a 5 5/8 inch diameter rock bit. A concentrated drilling mud mixture was used to minimize moisture content changes at the bore hole wall and an attempt was made to keep the drilling rate compatible with that at which pressuremeter tests could be conducted. The 5 5/8 inch

bit used with a stabilizer proved well suited to pressuremeter testing. Bore hole alignment problems were eliminated using this arrangement. In addition, the probe could be inserted in a slightly inflated condition ensuring the application of an all-round pressure on the probe by the drilling mud to counter the piezometric head on the measure cell.

3.1.3 Settlement Analysis

Several analyses were made using an axisymmetric finite element program originally developed by Wilson (1965). The analyses were limited to the linear elastic case with variations in moduli assigned to the fill and mesh size. In view of the foundation depth over which pressuremeter tests were conducted, the effect of the position of a rigid lower boundary on displacements had to be explored for this case. The finite element mesh used in the analysis is presented on Figure 8.

Variations were made in deformation moduli assigned to the fill material for the load application.

All loads had to be applied as equivalent nodal loads, as the program did not distribute gravity loading on an elemental basis. All loads were considered those acting on a one-radian segment of the mountain. Nodal loads were assigned on the assumption that the centroid of each element occurred at its mid-point. The deviation between this location and the actual location of the centroid is insignificant in this case. A unit

weight of 120 pcf was used in deriving nodal loads.

Load application was made in two stages to simulate actual construction. The stages are indicated on Figure 4. A modulus of 50,000 psf was assigned arbitrarily to the fill material. A variation in the fill modulus of one order of magnitude was found to produce a 1.3 inch variation in foundation settlement, the settlement decreasing with higher moduli for the fill. Placing a fixed boundary at the bottom of the Floral till, the depth to which moduli had been obtained with the pressuremeter, produced centerline settlement that differed by 5 inches from that obtained with a fixed boundary located in the Bearpaw shale. Because a reliable relationship was believed to be established between pressuremeter and laboratory determined moduli the lower fixed boundary was used in subsequent analysis.

A comparison of pressuremeter-determined moduli with laboratory data from Hamilton and Tao (1972) is shown on Figure 6. The pressuremeter moduli are seen to be double the laboratory figures with the exception of moduli obtained in the field at a very shallow depth. Little increase in deformation moduli with depth was observed for the Floral till. For this reason a single modulus was applied to the Sutherland till, this figure being double that reported by the above authors.

A load-settlement-time relationship was constructed for the centerline portion of the mountain and is shown

on Figure 5. The shape of the time-settlement curve for the period November 1969 to April 1970 was estimated as settlement readings were not available at the completion of the first stage of construction. Measured total settlement at the beginning of Stage II of construction was 8.5 inches. Predicted immediate settlement for the first stage of construction was 7.2 inches. Measured total and predicted immediate settlements for the period of the second stage of construction only are 22.5 and 15 inches respectively.

The following reasoning was used to establish what portion of measured total settlement could be considered immediate. Significant settlement continued for only a 4 month period following the end of construction. Over this period, 11 inches of settlement was recorded. Considering that 11 inches of time-dependent settlement occurred in 4 months under full load, it was reasoned that one-half of this amount would have occurred during the construction period had it lasted the same length of time. On this basis, the time-dependent portion of settlement that occurred during loading would be $1/2 \times 11 \times \frac{3.5}{4} = 4.8$ inches. Subtracting this from measured total settlement yields an immediate settlement totalling 17.7 inches. The predicted immediate settlement amounted to 15 inches for the second stage of construction. The results of the settlement analysis are indicated on Figure 4. The modulus profile used in

the study are indicated on Figure 6.

3.1.4 Summary

Deformation moduli obtained by the pressuremeter in the Floral till are essentially double the laboratory determined moduli. This relationship appears reasonable considering the possible sample disturbance encountered in obtaining specimens for laboratory tests.

The load-time-settlement relationship shown on Figure 5 is considered typical of overconsolidated soils. For the second stage of construction, measured total settlement during construction is 2/3 of ultimate total settlement. Using reasoning outlined in Section 3.1.3, measured immediate settlement is approximately 3/4 of measured total settlement for the second construction period.

The fact that the mountain was not constructed in a single stage complicates a generalization as to what portion of ultimate total settlement (excluding secondary consolidation) can be considered immediate. A desirable approach to the prediction of settlement or displacements of structures founded on these soils would be a realistic estimation of immediate settlement using pressuremeter moduli coupled with a known relationship between this and total ultimate settlement. This sort of approach can be refined only through field experience with these soils or with a theoretical application of a consolidation theory for unsaturated soils (Fredlund,

1972). A more practical approach to the prediction of settlement would be the one outlined above. It is likely that for relatively large loaded areas such as under large spread footings, belled piles or raft foundations, a sufficiently reliable relationship could be established between immediate and total (or time independent and time dependent) settlement. This relationship would apply only to specific soil types peculiar to a region and would be valid for only those conditions that approximate to the linear elastic case.

Where time dependent movement becomes dominant, settlement predicted on the basis of conventional consolidation testing may be more appropriate.

The engineering behaviour of partially saturated soils has been explored with an emphasis on the application of the principle of effective stress to these soils. There is very little documentation in the literature of cases that involve the deformation of these soils on a field scale. To the author's knowledge, Fredlund's theory is the only one for the prediction of the time dependent portion of deformation of these soils.

The settlement analysis described in Section 3.1.3 supports the validity of the deformation moduli obtained from the pressuremeter test. The test interpretation adopted for glacial tills appears reasonable.

It is realized that the analysis involves the assumption that the pressuremeter moduli are double the

laboratory moduli for the Sutherland till and that rather limited laboratory data were available for this till sheet. The deformation modulus can be considered to increase with depth however, in that each till sheet has a higher modulus than its overlying one. For this reason, straining in the Sutherland till is probably less significant than predicted by the finite element method. This statement is made on the basis of investigations into the depth to a rigid boundary in finite element studies made by Matheson (1971).

Even after provision has been made for consideration of the time-dependent portion of the total settlement a certain discrepancy remains between the calculated probable and the observed settlements. The calculations under-estimate the deformation by some 15 percent.

An explanation of this feature can be found in the following:

Mt. Blackstrap is a structure which has a factor of safety in the conventional limit-equilibrium sense of the order of 1.5. This is small when compared to the other structures analyzed in this study. It is obvious that appreciable parts of the foundation are stressed rather close to shear failure, if not already failed. The stress-strain behaviour of soils is markedly non-linear, with modulus decreasing at high stress levels and this is not considered in the linear elastic analysis.

The Mt. Blackstrap case history is important in

that it demonstrates the limits of the applicability of this approach.

3.2 Pile Load Test, Calgary

3.2.1 Site Description

The Calgary air terminal site, under construction at the time of writing was chosen for additional field studies. A large-diameter, cast-in-place, concrete pile had been load tested at the site in the course of design work undertaken by the Ministry of Transport and R. M. Hardy and Associates Ltd. The pile was well suited to this type of analysis in that shaft friction had been eliminated over most of the pile length and the pile was embedded in bedrock.

The site is located immediately north of the existing facilities at McCall Field. Stratigraphy at the site consists of a bouldery till underlain by the Paskapoo Formation, comprised at this location of clay shale, sandstone and siltstone strata. The siltstone stratum in which the pile is founded is considered weak and moderately fractured within the upper six to eight feet. A major transition in deformation moduli occurs within the clay shale. For financial reasons, coring was not undertaken at the pressuremeter test hole and direct confirmation of the siltstone-shale boundary could not be made.

Shaft friction was eliminated over all but the

lower 5 foot length of the pile. This was accomplished by using concentric steel housings through the till and a bond preventive feature in the bedrock above the 5 foot tip section. A load cell had been installed in the pile tip in an effort to indirectly measure shaft adhesion over the tip portion. Tell-tales had been installed to eliminate pile compression from deflection data. The test pile details are shown on Figure 9.

3.2.2 Field Test Program

Financial assistance for the program was provided by the Calgary office of R. M. Hardy and Associates Ltd. The investigation was carried out with the co-operation of the Ministry of Transport, Government of Canada.

A Failing 1500 rig was used in the drilling program. Wet drilling with a 5 inch carbide insert was used in conjunction with a stabilizer shaft to maintain bore hole alignment. Four pressuremeter tests were conducted in the bedrock at intervals shown on Figure 10. Test Hole 748 had been drilled in the summer months of 1973 in which coring and visual inspection had been carried out by R. M. Hardy and Associates Ltd. As indicated on Figure 10, the pressuremeter test hole was approximately 25 feet from Test Hole 748 and 38 feet from the pile. Pressuremeter tests had been performed at the site in Test Hole 748. Both probes produced modulus profiles of nearly identical patterns with regard to variation with depth. This feature was employed in arriving at

the inferred stratigraphy shown on Figure 10. Field data in sufficient detail was not available such that a comprehensive comparison of test results for the two probes could be made.

The pressuremeter tests were conducted using 4 to 6 increments of 15 - 20 psi, commencing at pressures considered roughly equal to original lateral ground stresses. At the 46 and 66 foot depths, significant volume change occurred with time after the application of a pressure increment. In these cases the immediate deformation was considered to occur up to the point at which volume change varied linearly with time. (See Figure B-2, B-4). Bedrock stratigraphy and the modulus profile are indicated on Figure 18.

3.2.3 Analysis of Pile Performance

Several finite element analyses were attempted varying the mesh size and material properties. Different assumptions were made as to the nature of bonding over the lower 5 feet of the pile. The analyses are depicted graphically on Figures 11 and 12.

Contact pressures across the pile tip were considered uniform and nodal loads were calculated from a uniform pressure over a one radian segment of the pile. Wilson's axisymmetric program was used assuming linear elasticity and single step load application.

Loads were applied at the top of the bonded embedment portion of the pile to simulate prototype loading

and to avoid stress concentrations at the pile tip. Pile loading was limited to 350^K, considered a design load. The pile had been load tested to 900^K in several stages. Measured load-deflection data are presented on Figure 13.

The results of the analyses are presented on Figures 14 and 15.

3.2.4 Summary

A linear elastic analysis of large-diameter pile performance is considered valid up to design or working loads. Such loads are usually defined on the basis of allowable bearing stresses, pile stresses and/or pile deflection criteria. The validity of the linear-elastic analysis is substantiated by the load-deflection data of Figure 13 and by Osterberg and Gill (1973).

It is obvious that assumptions pertaining to bonding at the pile-bedrock interface are critical in predicting pile deflection, even under working or design loads. The measured load-deflection curves suggest that peripheral bond destruction occurred at an axial load of the order of 100^K and that a cohesive form of bonding was not restored upon unloading. Osterberg and Gill (1973) suggest that shear stresses are set up over the embedment in accordance with a strength envelope having a cohesion intercept and an angle of shearing resistance of the order of 30 degrees. Their findings are based on parametric finite element studies.

Measured pile performance for this case is considered best simulated with mesh number 1. Complete bonding is assumed to occur over the embedment portion of the pile. Under an applied load of 350^K, the calculated or predicted tip displacement is 0.21 inches. Measured displacement was 0.30 inches.

The overall dimensions for mesh mumber 1 are considered adequate, as a larger mesh (by a factor of two) produced the same displacements. The dimensions of mesh number 1 are considerably smaller than those recommended by Osterberg and Gill (1973).

The above authors concluded, on the basis of parametric finite element analyses, that the presence of a softened layer immediately below the pile tip for the case of embedded piles does not significantly affect tip displacement, the reason being that such a condition has little influence on load transfer over the pile embedment. This observation is supported by analyses performed in the course of this study. Confirmation on the basis of field behaviour is not possible from this one case.

Because of uncertainties in detailed bedrock stratigraphy, a number of analyses were conducted for which the depth of relatively low modulus material was varied below the pile tip. The finite element analyses would suggest that socketted piles, for which the embedment to diameter ratio is of the order of 3, develop their capa-

city under working loads largely by mobilization of shaft friction. A variation in the thickness of the low modulus siltstone from 15 to 40 inches produced a variation in tip displacement of only .05 inches. The distribution of shear stress over pile embedment, as determined using mesh number 1 is indicated on Figure 13. The pattern of shear stress distribution is reasonable and variation in magnitude is very small. The axial load distribution obtained for mesh number 1, shown on Figure 17, is reasonable and compares favourably with the measured distributions reported in the literature (Freeman et al, 1972).

Discrepancies between observed pile deflection and calculated displacements are possibly a function of the degree to which the stressed rock will undergo time dependent deformation, the presence of disturbed material below the pile tip and the anisotropy of the deformation modulus of layered media. Although the time dependent portion of pile displacement was not determined in the course of the actual load testing, the observed deflections indicated on Figure 13 suggest this mode of movement was significant, even at working loads.

The effects of disturbed or softened material at the pile tip are difficult to account for and, on the basis of the analysis done in this study, are not very significant for piles that develop most of their capacity through mobilization of shearing resistance over

the embedded portion of their shafts. Disturbed or softened material at the tip would have a more obvious effect on the behaviour of piles which rely primarily on end-bearing.

Burland and Cooke (1974) present an interesting method for predicting the behaviour of belled or under-reamed piles constructed in stiff clay. Their observations would confirm the validity of the use of linear-elastic methods in the prediction of pile behaviour up to working loads.

For design purposes, the results of this study would suggest that working loads for socketted piles may be assigned on the basis of displacement criteria. One such method may include the prediction of pile movement based on finite element analyses using pressuremeter moduli, the multiplication of this predicted movement by some factor to account for time dependent deformation and the assigning of a design load on the basis of acceptable pile displacement. The empirical correction factor would also account for the mechanism of bonding along the pile shaft and the effect of the softened zone below the tip. Such correction factors would have to be derived from load tests for a particular construction technique and geological environment.

3.3 Settlement Study, Canada Malting Storage Silos, Calgary

3.3.1 Site Description

Reinforced concrete storage silos located in Southeast Calgary were chosen as a site for field studies of the pressuremeter application in granular soils. The silos are of the order of 120 feet in height and are founded on a concrete raft having dimensions of 75 by 130 feet in plan and a thickness of 5 feet. The silos are used for grain storage and well-suited for settlement study purposes.

Information regarding soil stratigraphy and general geology of the area as well as settlement records were obtained from the files of R. M. Hardy and Associates Ltd., Calgary, Alberta.

The site is located on a wide, flat, alluvial terrace within the present Bow River valley.

The terrace is overlain with a layer of coarse, well-rounded gravel together with sand, silt, cobbles and occasional boulders. The gravel has been deposited directly onto the Cretaceous bedrock which exhibits an undulating surface that may be the consequence of preglacial incision or erosion. Bedrock is of the Paskapoo Formation and is comprised of siltstone and clay-shale interbedded with sandstone occurring in layers and lenses. Bedding is essentially horizontal.

More detailed information on the nature of the bedrock strata was not obtained in the course of the fieldwork. Coring of these members was not undertaken and

visual examination of the cuttings, during drilling, provided little opportunity for differentiation of the strata which were encountered.

Stratigraphic variations in the horizontal and vertical directions below the structure were relatively minor. The soil and bedrock profile indicated on Figure 20 is believed to be representative of conditions over the area bounded by the structure and was used in the settlement analysis.

3.3.2 Field Test Program

Test drilling was performed using both cable tool and Failing 1500 rigs. Initial drilling was done with the cable tool drill which is considered better suited for work in coarse gravels. Pressuremeter tests had not been performed locally in gravels, and hence a technique had to be established for testing in such soils. The probability of caving in the course of testing was considered. Casing was advanced to within 5 feet of the bottom of the test hole to prevent caving above the probe during testing. Pressuremeter tests were performed immediately below the casing. In addition the test hole had to be cased to the bedrock prior to tests in that material. The groundwater level was within the gravel layer, approximately 20 feet from the surface.

Bore hole disturbance was considered significant when drilling with the cable tool drill, the bit being advanced by impact of the tool. The dynamic loading in

the vicinity of the bore hole was considered to seriously affect the deformation modulus of the gravel. Test holes put down in the course of a foundation investigation prior to construction of the silos revealed the gravel to be in a relatively dense state. Dynamic loading in the course of test drilling would likely lead to a reduction in the relative density of the gravel at the bore hole and have a corresponding effect on measured moduli.

For these reasons, additional test holes were put down with a rotary drill, the Failing 1500. Difficulties were experienced but two test holes were considered suitable for testing. The test hole locations are indicated on Figure 20. The test holes were located such that representative soil and bedrock conditions could be encountered without having to perform the tests within the zone of material influenced by the building load. A 6 1/4 inch diamter rock bit was used in conjunction with relatively heavy drilling mud. Caving of the test holes in the course of testing above the water table was not experienced. Tests were performed at the same elevations in Test Holes 2 and 3 for purposes of comparison. In addition, tests were conducted to the 75 foot depth in Test Hole 1.

3.3.3 Test Results

Time-volume change and pressure-volume change curves obtained at this site are presented in Appendix B.

The results appear to confirm the adverse effect

the cable tool drill had on the soil surrounding the bore hole. The time-volume change curves were considered invalid within the gravel layer in Test Hole 1. Tests conducted in the rotary-drilled holes were considered valid.

The production of a borehole having a relatively smooth or uniform wall is not possible in a coarse gravel. The test results do not appear adversely affected by this bore hole condition, though the degree to which this effect can be evaluated is not clear. The non-uniformity of the bore hole wall is not reflected in volume change behaviour. Direct bore hole diameter measurements were conducted in one of the test holes, revealing variations of the order of an inch or two over the length of the probe. It is likely that the existence of excessively large irregularities or voids would result in rupturing of the outer membrane in the course of testing. Volume change behaviour associated with the filling of bore hole voids or irregularities would produce a distinctly non-linear pressure-volume change relationship. This was not the case at the studied site.

For the above reasons, the tests conducted in gravel are considered acceptable with regard to the performance of the probe.

3.3.4 Settlement Analysis

The load-time-settlement relationship indicated on Figure 19 was obtained for the period of November 25, 1971 to December 14, 1973. Settlement observations were obtained by means of conventional level surveys referenced to a deep bench mark. Settlement plugs were located at a number of interior points at the base of the silos which had been constructed monolithically with the foundation raft. The settlements indicated on Figure 19 are average centerline settlements in the longitudinal direction of the structure.

Indicated on Figure 21 are the finite element mesh and representative soil stratigraphy and deformation parameters used in the analysis. A plane strain, linear-elastic program utilizing constant strain, triangular elements was used in the analysis. The program was originally developed by Wilson (1965).

In view of the fact that pressuremeter tests were conducted to only the 75 foot depth, several analyses were conducted.

An analysis was made using the measured moduli and assuming that a constant value was applicable from the 75 foot depth to the base of the mesh. Another analysis was made assuming a linear increase in modulus with depth as indicated on Figure 21. The deformation moduli were considered to increase in magnitude in accordance with the relationship defined by test results at the 40, 60

and 75 foot depths.

On the basis of a large number of triaxial and oedometer tests, a relationship between deformation moduli versus depth was established by Janbu (1963). However, apparent increase in modulus with confining stress does not correlate well with this relationship. Janbu's parameters for rock or moraine produce essentially a constant modulus with depth. The increase with depth observed at the storage silos must be more a function of stratigraphic changes than confining stress. Consequently, a relationship for moduli versus depth based on Janbu's work could not be utilized.

Values for Poisson's ratio of 0.30, 0.35 and 0.25 were chosen for gravel, bedrock and concrete elements respectively. Foundation loads were applied in a single step to weightless media in the finite element simulation.

Although the silos were filled in stages, a reasonable approximation used in the analysis consisted of loading to 50 percent and to 100 percent of full load. The calculated settlements are considered immediate and are indicated for illustrative purposes as time-independent on Figure 19.

3.3.5 Summary

A settlement analysis utilizing the deformation moduli determined with the pressuremeter at the site of the storage silos correlates reasonably well with observations.

The best correlation was obtained using the linear increase in moduli with depth, as indicated on Figure 21. The analysis utilizing a constant modulus of 30,000 psi from the 75 foot depth resulted in settlements that exceeded the observed by approximately 10 percent. The linear increase in moduli as defined by the relationship obtained at the 40, 60 and 75 foot depths was considered realistic. The results of the analysis using this relationship are indicated on Figure 19.

For this case it was not possible to differentiate between immediate and total, observed settlement using rationale presented in Section 3.1.3. The shape of the loading curve had to be estimated during the second stage and it is considered that insufficient readings were obtained just after full loading to accurately define the time-settlement curve.

The calculated displacement can be considered to over-estimate the probable immediate settlement and to correlate closely with observed total settlement.

Although the moduli obtained with the pressuremeter relate reasonably well to the foundation performance, the test results in the gravel do not compare well with the results of plate bearing tests performed in similar materials in downtown Calgary by Clark and Robinson (1972). Initial tangent moduli obtained by the above authors are more than double those obtained with the pressuremeter. The pressuremeter moduli appear to compare more

favourably with the secant modulus from plate tests, as observed by Meigh and Greenland (1965). Differences in the nature of the test methods and the stress-deformation characteristics of the gravel are probably factors of significance related to differences in moduli.

CHAPTER IV

A DISCUSSION OF DEFORMATION MODULI

4.1 Introduction

Deformation moduli are required in analysis in various geotechnical problems, primarily in foundation engineering. Stress-displacement analyses in this field have been considerably improved with the development of the finite element method. Fundamental to the reliability of these analyses is the use of realistic and representative deformation moduli. Insitu moduli obtained by the pressuremeter are considered to better represent field conditions than moduli obtained by conventional means in the laboratory. Although non-linear analyses are possible, factors of safety commonly employed in foundation engineering result in working stress levels that are of the order of 1/2 to 1/3 of shear strengths. For this reason a linear-elastic approach to stress-displacement analyses is justified and deformation moduli obtained by the pressuremeter are potentially applicable.

A rather pragmatic approach to the application of deformation parameters obtained by the pressuremeter is pursued in this study. The validity of the test results is considered better assessed from a case-history approach than from a solely theoretical one. Considerations regarding stress paths taken in the pressure-

[illegible]

[illegible]

[illegible]

[illegible]

meter test, anisotropy, the influence of confining stresses and the volume of material that is stressed in the course of testing are not elaborated on in this study. Arguments regarding the theoretical relevance of the test results to the prediction of the field behaviour of foundation soils are not advanced. The results of case histories presented here and elsewhere suggest that deformation parameters at least, obtained from the pressuremeter test, relate reasonably well to the field behaviour of the tested materials.

The low degree of confidence with which moduli obtained from conventional laboratory specimens can be applied has been demonstrated. Morrison (1972) and Dejong (1971) report that moduli obtained in the laboratory are smaller by as much as an order of magnitude than pressuremeter moduli. The ratio of lab to field determined moduli for the Saskatchewan tills is 1:2, the laboratory moduli having been measured in both oedometer and triaxial tests. Pressuremeter moduli have also been found to be lower than laboratory moduli at a given location. (Dixon, 1970).

There is some question as to what modulus of deformation is measured in the pressuremeter test, particularly with regard to that in granular media. The question arises from the fact that soil adjacent to an expanding cavity is under failure deviator stresses while soil beyond the region of plastic deformation is

under stresses below failure in magnitude, approaching zero at a distance of approximately 20 radii of the bore hole from its centerline.

The findings of Meigh and Greeland (1965) and Clark and Robinson (1972), commented on in Section 4.3, suggest the pressuremeter modulus does not compare with an initial tangent modulus from plate bearing tests. It is possible that the modulus relates better to a second loading modulus determined for laboratory specimens, in that these moduli are considered to relate reasonably well (for cohesive soils) to foundation deformations. As pointed out elsewhere in this thesis, a purely pragmatic approach to the validity of the pressuremeter modulus and its relevance to foundation settlement in the geological environment of Western Canada is adopted in this study.

In the writer's opinion the pressuremeter can be considered a useful tool with regard to the investigation of the insitu deformation properties of glacial till and soft bedrock. The deformation moduli of tills and soft bedrock obtained with a probe of this design can be used with confidence in the prediction of immediate deformation of these materials.

Menard (1965) and others have developed empirical methods for the derivation of the bearing capacity of deep and shallow foundations and strength parameters of various soils that are based solely on several aspects

of pressuremeter test results. As mentioned earlier in this study, this approach to employment of the instrument is not recommended.

Only meagre data has been published concerning the validity of strength parameters obtained from the test. The theoretical analyses are sound with regard to the evaluation of strength parameters and their application as documented in published case histories is increasing.

4.2 Tests in Granular Soils

The results of tests performed in the coarse dense gravel at the Canada Malting Co. site in Calgary are considered reasonable for the condition of first time loading in that they compare with moduli reported in Lambe and Whitman (1969). They suggest a secant modulus to 1/2 of peak deviator stress of 5,000 and 15,000 psi for angular-breakable and hard-rounded particles respectively. The moduli are reported for a confining pressure of 1 atmosphere.

Clark and Robinson (1972) report on a field test program performed in Calgary, Alberta. The investigation involved the determination of the deformation properties of a gravel deposit underlying the Calgary Tower. Plate bearing and seismic tests were conducted. The results of the seismic tests were used to evaluate the variation with depth and the anisotropy of Poisson's ratio and the elastic modulus of the gravel. Anisotropy of these properties was related to a preferred particle

orientation that was established by a petrofabric analysis. The authors quote initial tangent moduli for first loading of 25,000 and 72,000 psi for a 12 inch plate and 25,000 psi for an 18 inch plate bearing test. Average deformation moduli were back-figured from load-settlement data of the Calgary Tower. A closed-form solution was used and the moduli were shown to increase with increasing foundation load to 40% of maximum applied dead load. To what degree this behaviour is a function of the gravel layer is speculative in that the thickness of the gravel layer was roughly 1/3 the width of the ring foundation.

Vesić and Clough (1968) have shown the deformation modulus of sand to increase at a variable exponent of confining stress. They also indicate a variation in the relative compressibility of sand with mean normal stress. The ratio of shear modulus to initial shear strength or rigidity index is shown to be roughly inversely proportional to the square root of confining pressure.

Vesić (1972) reports pressuremeter data from a test at the 45 foot depth in a medium dense sand. The deformation modulus and angle of shearing resistance were calculated to be 1,500 psi and 33 degrees respectively. Taking volumetric straining into account, repeated calculations resulted in an angle of shearing resistance of 37 degrees. The deformation modulus was shown to be insensitive to the consideration of volume change within the zone of

of plastic deformation around an expanding cavity. The angle of shearing resistance was shown to be highly sensitive to these volume changes, requiring that they be estimated to an accuracy of 0.1%. Calculations of Ø from pressuremeter tests using Vesić's analysis results in a conservative estimate when performed with no regard to volume change.

Gibson and Anderson (1961) report angles of shearing resistance for a medium to fine, uniform sand of 37 to 49 degrees. They report a deformation modulus of 2,700 psi for the same soil. They do not consider their theory pertaining to sands to be entirely satisfactory, in that they had to assume linear elastic behaviour below yield.

Calhoon (1969) reports on the results of a large number of pressuremeter investigations in the United States. The results are presented in the form of settlement predictions based on pressuremeter test results and data from the SPT test. He concludes that the pressuremeter results relate better to settlement prediction for a variety of soils than do SPT data. The information he presents is of limited value in that the actual moduli are not reported.

4.3 Tests in Soft Bedrock

Dixon (1970) reports deformation data obtained from pressuremeter tests in California. The tests were conducted in interbedded sandstone and shale of the Castaic Formation. Rebound moduli were measured at the site to

provide an indication of the degree of fracturing. The ratio of rebound to compressive moduli varied from 1.5 to 10 and was considered indicative of moderately to heavily fractured rock. This fact was verified by visual inspection. Modulus values for the sandstone were consistently lower than for the siltstone, averaging 1/2 the value for the latter. Compressive strengths and densities were also lower for the sandstone. Moduli from the pressuremeter tests were lower than those from laboratory samples but were considered more representative of the insitu properties and subsequently used for design purposes in the construction of a surge chamber.

Pressuremeter tests were performed by Dixon in clayey siltstone near Los Angeles. He reported moduli varying from 5,000 to 18,000 psi with rebound moduli indicative of virtually intact rock.

Morrison (1972) obtained deformation moduli varying from 13,000 to 130,000 psi for fractured and intact shale respectively of the Edmonton Formation.

In the course of field work for this study, tests were conducted in soft bedrock of the Paskapoo Formation in Calgary, Alberta. Deformation moduli as low as 9,800 and 12,000 psi were obtained for weathered shale and siltstone respectively. Representative values for shale below the weathered strata of this formation are of the order of 25,000 to 65,000 psi.

The best documented correlation study of deformation

and strength parameters from several sources is that of Meigh and Greenland (1965). Tests were performed in Coal Measure mudstone and sandstone, Keuper marl and Bunter sandstone. Deformation moduli were obtained in the field from plate bearing and pressuremeter tests. Strength tests were performed with the pressuremeter and on drill cores, open-drive and block samples. The results of the study in general are summarized below:

1) Pressuremeter tests in the Coal Measure sediments yielded significantly lower deformation moduli below the ground water level than above it.
2) In a number of cases, air-flushed drill holes yielded higher moduli than water flushed.
3) Deformation moduli from the pressuremeter test appear to compare favourably with a "secant" modulus obtained from plate bearing tests. Initial moduli from the plate bearing test are significantly higher than moduli from the pressuremeter.
4) The undrained shear strength of very soft, very closely fissured siltstone, was determined from pressuremeter, plate bearing and triaxial tests on block samples. For this material, all three tests yielded essentially the same strength.
5) Shear strengths from the pressuremeter tests in the Coal Measure sediments were significantly higher than the results of triaxial tests.
6) Typical deformation moduli obtained from pressure-

meter tests are listed below. Moduli are given in pounds per square inch.

Silty Mudstone:	7,000 - 21,000 (above GWL)
	1,400 - 6,000 (below GWL)
Keuper Marl:	
(thin layered sandstone, siltstone)	3,000 - 37,000
Weathered Sandstone:	9,500

4.4 Summary

Deformation moduli obtained from pressuremeter tests in general are significantly less than initial tangent moduli for first loading from plate bearing tests. They probably relate better to moduli determined at significant plate movement, a form of secant modulus. It is likely that the results of pressuremeter tests in gravels correlate with the relative density of these soils. To the degree that pressuremeter tests can be easily "standardized" this test could prove more effective than the SPT in the evaluation of the insitu properties of granular soils.

Deformation moduli obtained from the pressuremeter tests in soft bedrock in the Calgary area appear reasonable. They are somewhat lower than modulus values obtained for similar sediments of the Edmonton Formation by Morrison (1972).

The influence of discontinuities is believed evident from the tests performed at the 50 and 60 foot depths at

the Canada Malting Co. site in Calgary. The modulus value at higher stress levels is seen to exceed that at lower levels. These are represented as initial and secondary moduli in Figure 16 and 17, Appendix B. The influence of crack closure is considered separable from the effect of the restoration of initial ground stresses. An examination of the results of a test at the 75 foot depth at the same site suggests the original ground stress was approximately 70 psi. The inflection in the pressure-volume change curves at the 50 and 60 foot depths occurred at pressure of 90 and 85 psi respectively. Linear behaviour is indicated prior to and after the inflection point, but this need not represent crack closure. The restoration of original horizontal ground stresses by the pressuremeter is a distinctly non-linear process.

CHAPTER V

CONCLUSIONS

The U of A probe constructed in the course of this study was found well suited to the measurement of the deformation moduli of glacial till, coarse gravel and soft bedrock. The instrument is not considered suitable for the determination of the deformation properties of intact rock having deformation moduli of the order of 200,000 psi or more. The measurement of volume change during testing in these materials would require a more sensitive measuring system than can be attained with a probe of this design.

Strength tests with the probe require high rates of fluid flow to the measure cell of the instrument. It is likely that such tests would be difficult to perform in relatively soft soils having the consistency of lightly overconsolidated lake sediments or normally consolidated soils. Strength testing in these materials would require fluid flows in excess of that obtainable with a probe of this design or the use of very small pressure increments, a complicating factor.

The preliminary laboratory trials that were conducted to assess the operation of the pressure reduction valve on the probe indicated a need for a different valve, such as the solenoid type. The trial and error procedure presently required in the setting of the reduction valve for testing at depth in a dry hole is not adequate. Utilization of an electric solenoid

valve as outlined in Section 2.3 would solve this problem.

The results of the field studies suggest that deformation moduli obtained with the 5 inch probe in the Saskatchewan glacial till and the Paskapoo Formation are valid. The moduli can be used with confidence in the prediction of immediate settlement of structures founded in these materials.

Deformation moduli of the Saskatchewan tills obtained with the pressuremeter are approximately twice the magnitude of laboratory determined moduli. For the case of the Saskatchewan tills, immediate settlement constitutes 2/3 to 3/4 of total settlement. The time-settlement relationship for Mt. Blackstrap is considered typical of that for overconsolidated soils.

The working capacity of end-bearing drilled piles in soft bedrock can be estimated on the basis of a linear-elastic stress-displacement analysis incorporating pressuremeter-determined deformation moduli. Pile tip displacement was predicted within 30% using moduli measured in the horizontal direction. The designation of design or working loads for end bearing piles founded in soft bedrock can be made on the basis of deformation criteria alone. For piles founded on bedrock having moduli in excess of 100,000 psi, working capacity should be assessed on the basis of allowable stresses in the structural member.

The assumption of complete bonding at the concrete rock interface for embedded end-bearing piles will lead to an underestimation of tip displacements. These displacements are over-

estimated for the assumption of no rock-concrete bonding. It is likely that bonding is incomplete at relatively small shaft displacements, that shear stresses at the interface are mobilized and that these shear stresses are not uniformly distributed along the embedded shaft. The finite element simulation of a more realistic load transfer mechanism requires the use of special elements at the pile surface, such as those used by Osterberg and Gill (1973). The parametric study done by the above authors supports the validity of the linear-elastic analysis incorporating complete bonding over pile embedment in estimating working loads.

Pile tip displacements are relatively insensitive to variations in Poisson's ratio for bedrock. A realistic value is considered to be 0.35.

Displacements predicted on the basis of the pressure-meter-determined moduli were in each field study less than measured displacements. For the case of the Saskatchewan tills, a rigorous assessment of the time-dependent portion of settlement was beyond the scope of this study. The separation of time-dependent and immediate settlements made on the basis of judgement outlined in Section 3.1.4 resulted in what is considered a reasonable estimation of each mode of deformation. A detailed evaluation of settlements associated with each mode would be required before one could consider the effects of anisotropy of the modulus of deformation in the horizontal and vertical directions. The anisotropy factor would be less dominant for the case of glacial tills in comparison with that

of sedimentary bedrock, from the point of view of structure of the materials. The more structurally isotropic nature of glacial till would lead one to expect a small variation in deformation moduli in the horizontal and vertical directions. The stratified nature of local bedrock strata would suggest a significant variation in moduli measured in different directions. An evaluation of the anisotropy factor is complicated by the existence of discontinuities within a bedrock mass and the effect of these on deformation moduli. In addition, there is a probable anisotropy of Poisson's ratio for layered bedrock, though the deformation modulus is not especially sensitive to this ratio. The anisotropy factor may be more relevant to the prediction of pile performance than to the prediction of displacements under large loaded areas. The existence of relatively large horizontal compressive stresses below a pile tip is a complicating factor in the assessment of the effect of structural anisotropy. For the case of the Calgary test pile, it is suggested that a definition of the load transfer mechanism is more relevant than isotropy to the prediction of pile performance using methods outlined in this study.

Pressuremeter moduli obtained in coarse, dense gravel appear reasonable and can be employed in finite element studies for the prediction of building settlement on these materials. The moduli for the gravel deposit tested at Calgary appear conservative when compared to results in the literature but can be considered to relate realistically to foundation performance.

VOLUME CHANGE OF THE MEASURE CELL
FOR INFLATION PRESSURES OF 6, 9 PSI

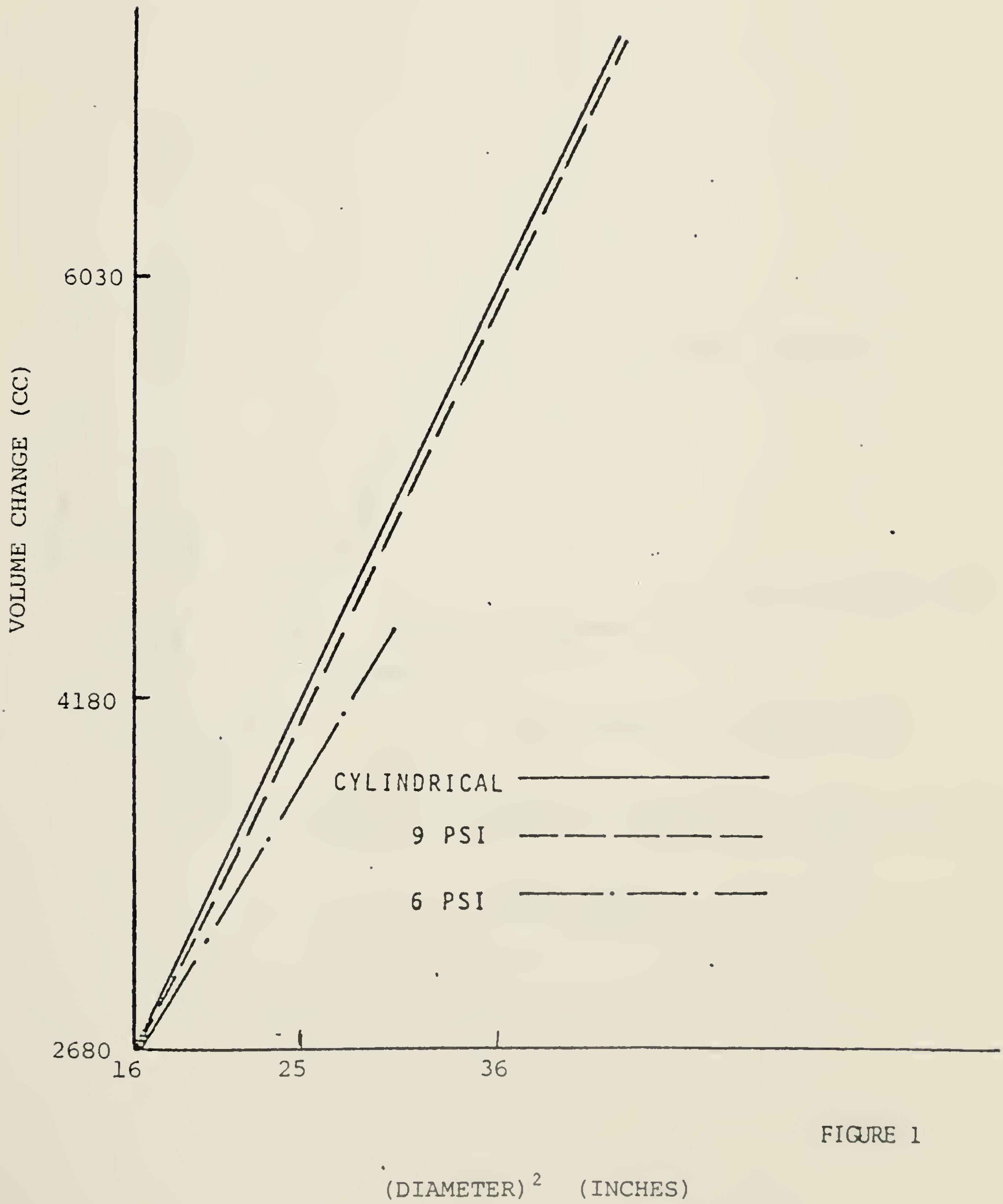


FIGURE 1

RATE OF VOLUME CHANGE OF MEASURE CELL

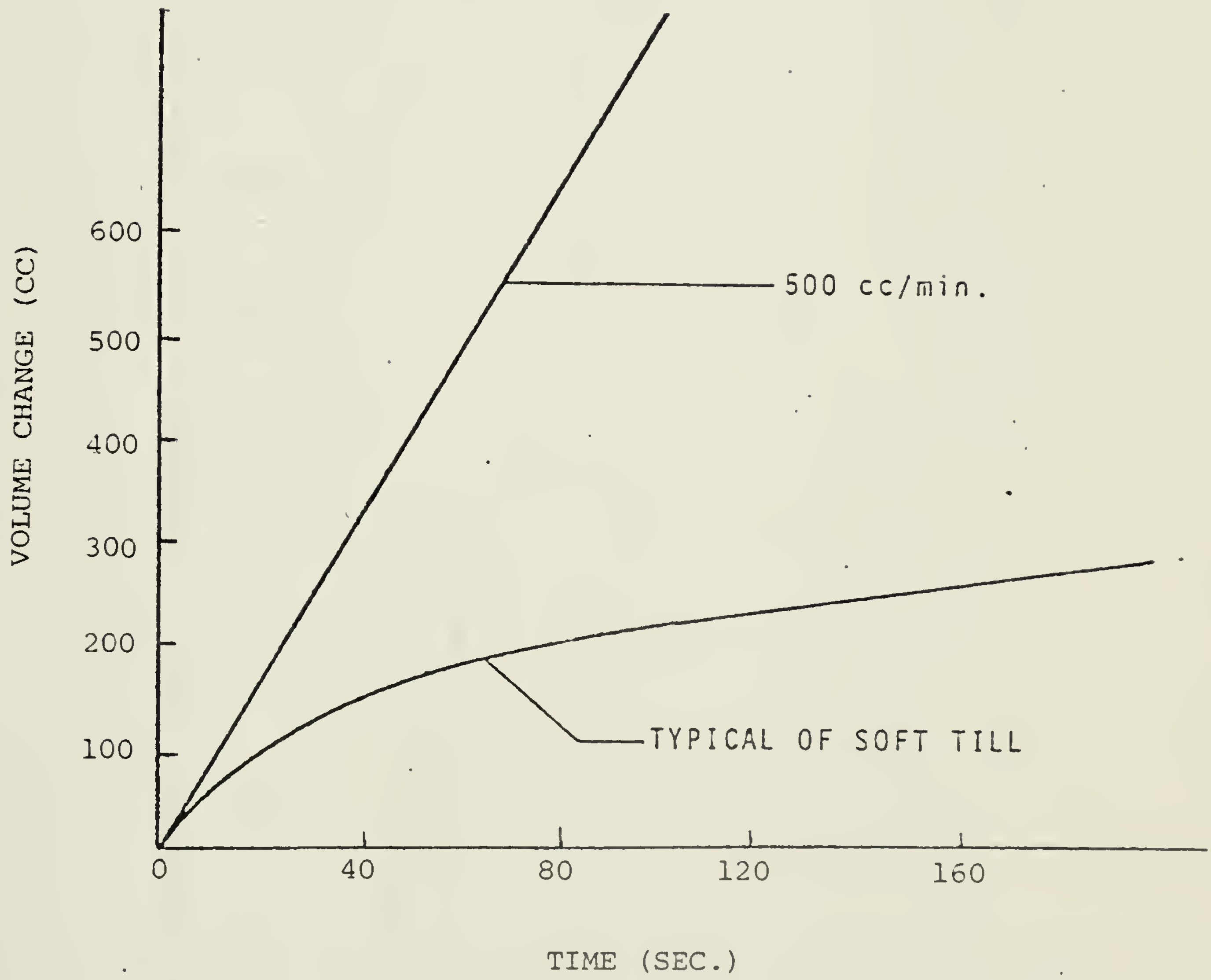


FIGURE 2

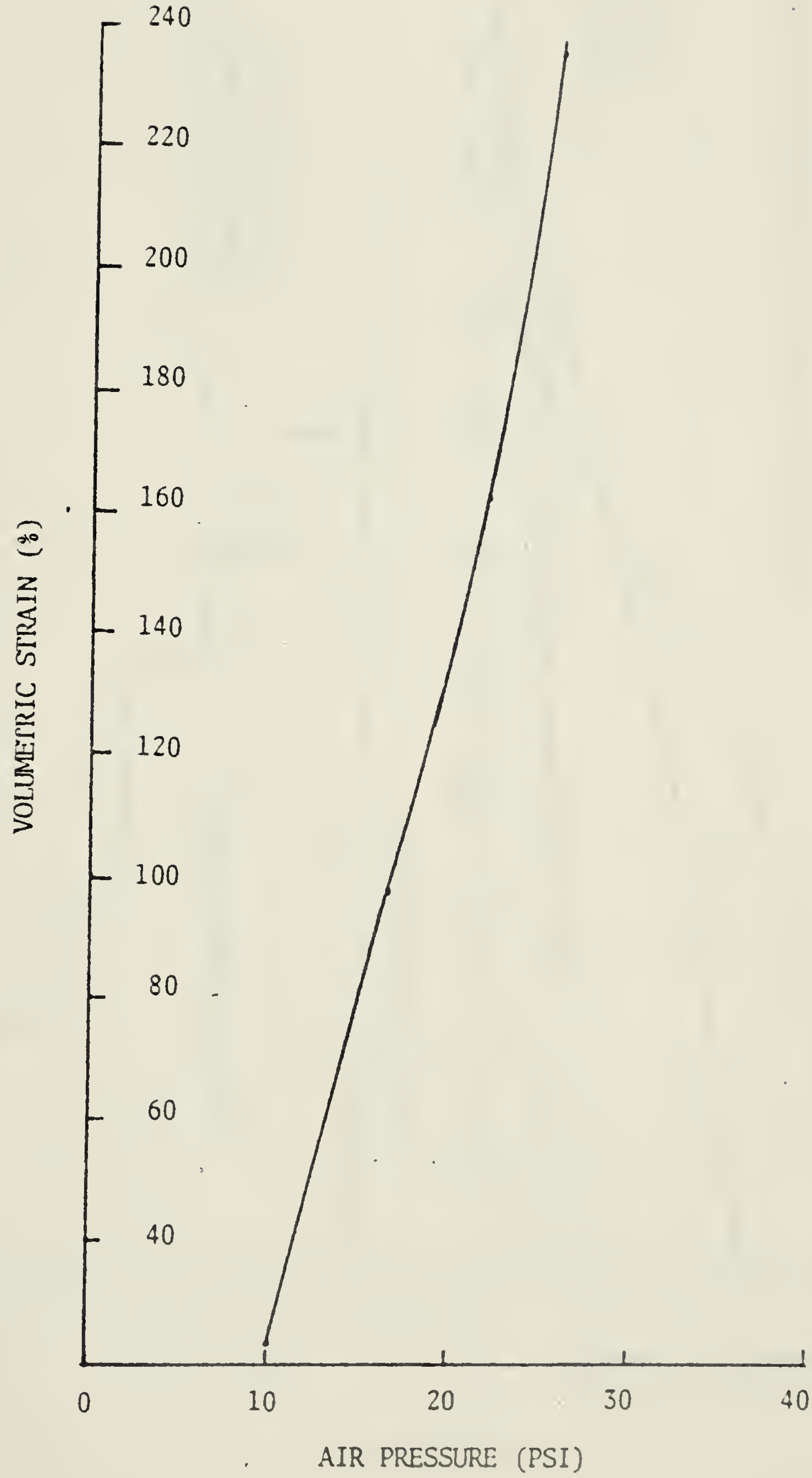
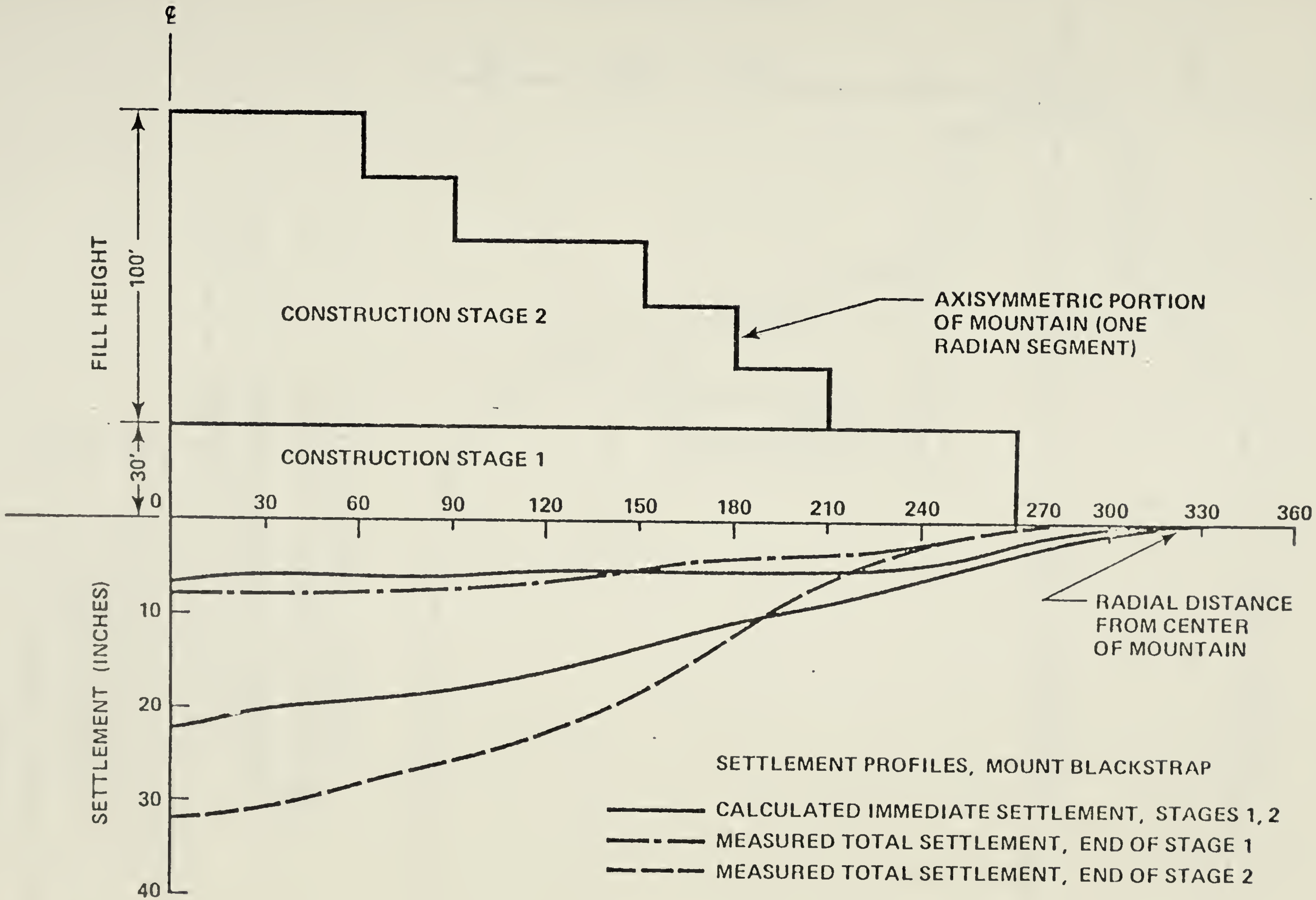
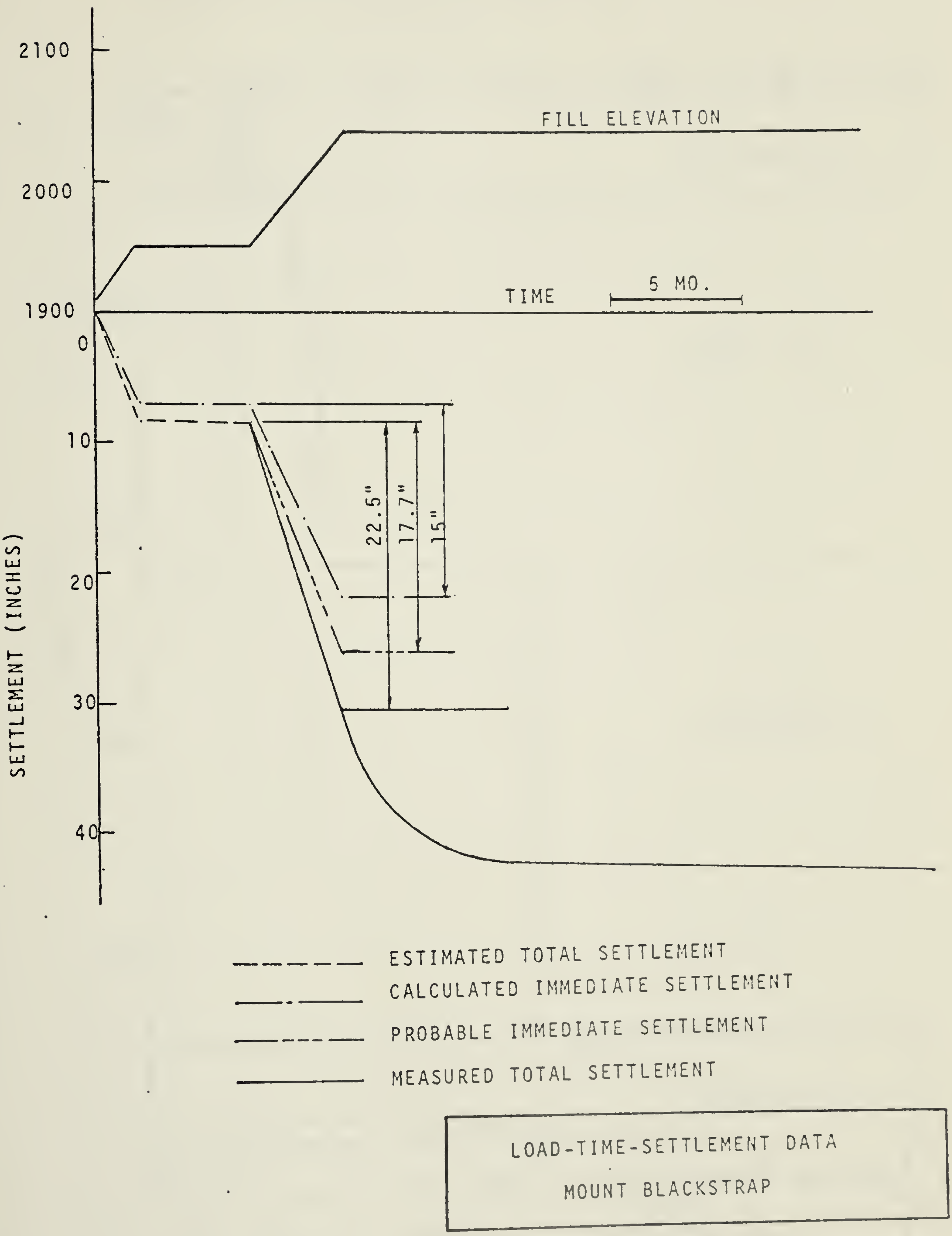

240
220
200
180
160
140
120
100
80
60
40
VOLUMETRIC STRAIN (%)
0
10
20
30
40
AIR PRESSURE (PSI)

FIGURE 3



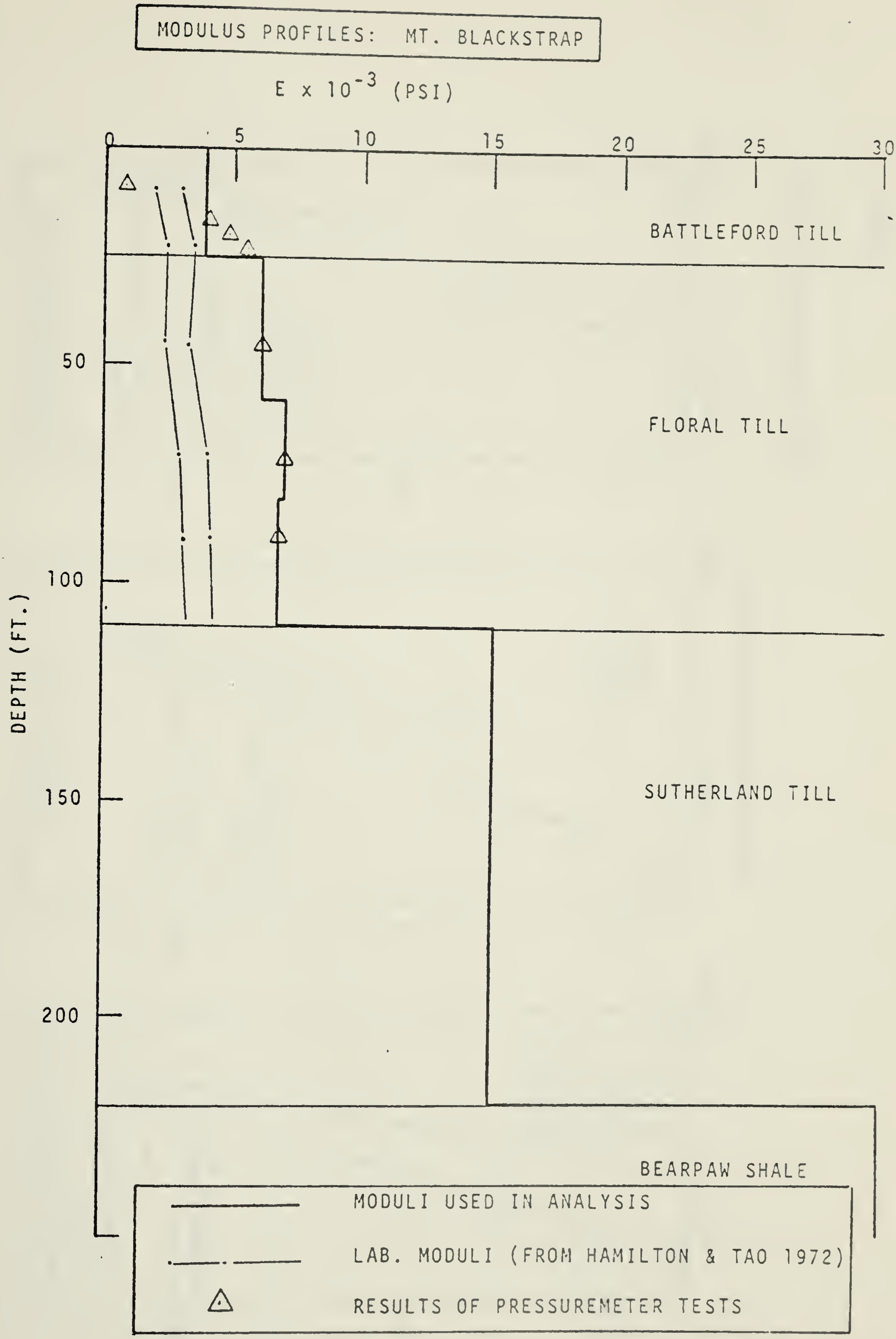
℄
FILL HEIGHT
100'
30'
CONSTRUCTION STAGE 2
CONSTRUCTION STAGE 1
AXISYMMETRIC PORTION OF MOUNTAIN (ONE RADIAN SEGMENT)
0
30
60
90
120
150
180
210
240
270
300
330
360
RADIAL DISTANCE FROM CENTER OF MOUNTAIN
SETTLEMENT (INCHES)
10
20
30
40
SETTLEMENT PROFILES, MOUNT BLACKSTRAP
CALCULATED IMMEDIATE SETTLEMENT, STAGES 1, 2
MEASURED TOTAL SETTLEMENT, END OF STAGE 1
MEASURED TOTAL SETTLEMENT, END OF STAGE 2

FIGURE 4



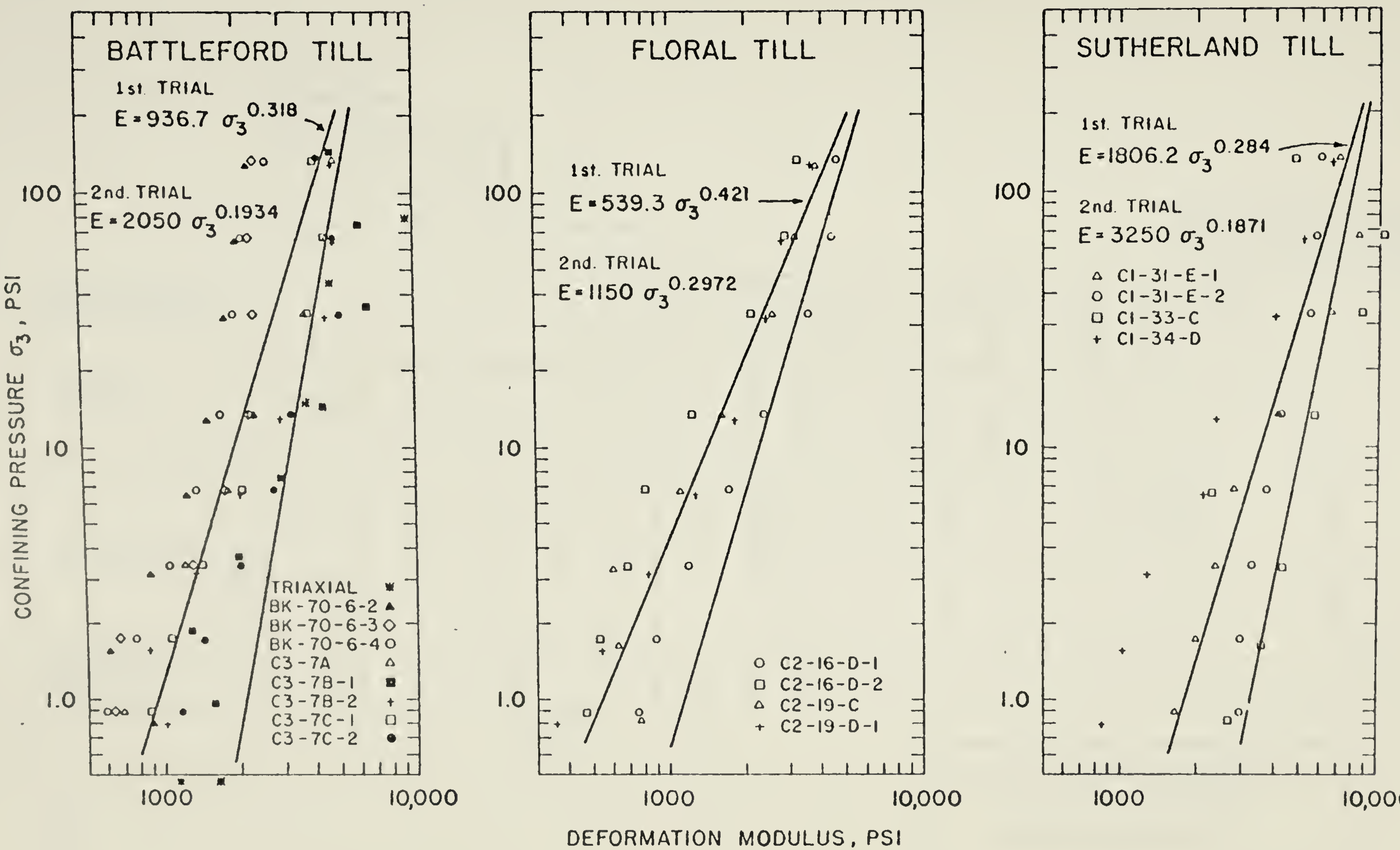
2100
2000
1900
0
10
20
30
40
SETTLEMENT (INCHES)
FILL ELEVATION
TIME
5 MO.
22.5"
17.7"
15"
ESTIMATED TOTAL SETTLEMENT
CALCULATED IMMEDIATE SETTLEMENT
PROBABLE IMMEDIATE SETTLEMENT
MEASURED TOTAL SETTLEMENT
LOAD-TIME-SETTLEMENT DATA
MOUNT BLACKSTRAP

FIGURE 5



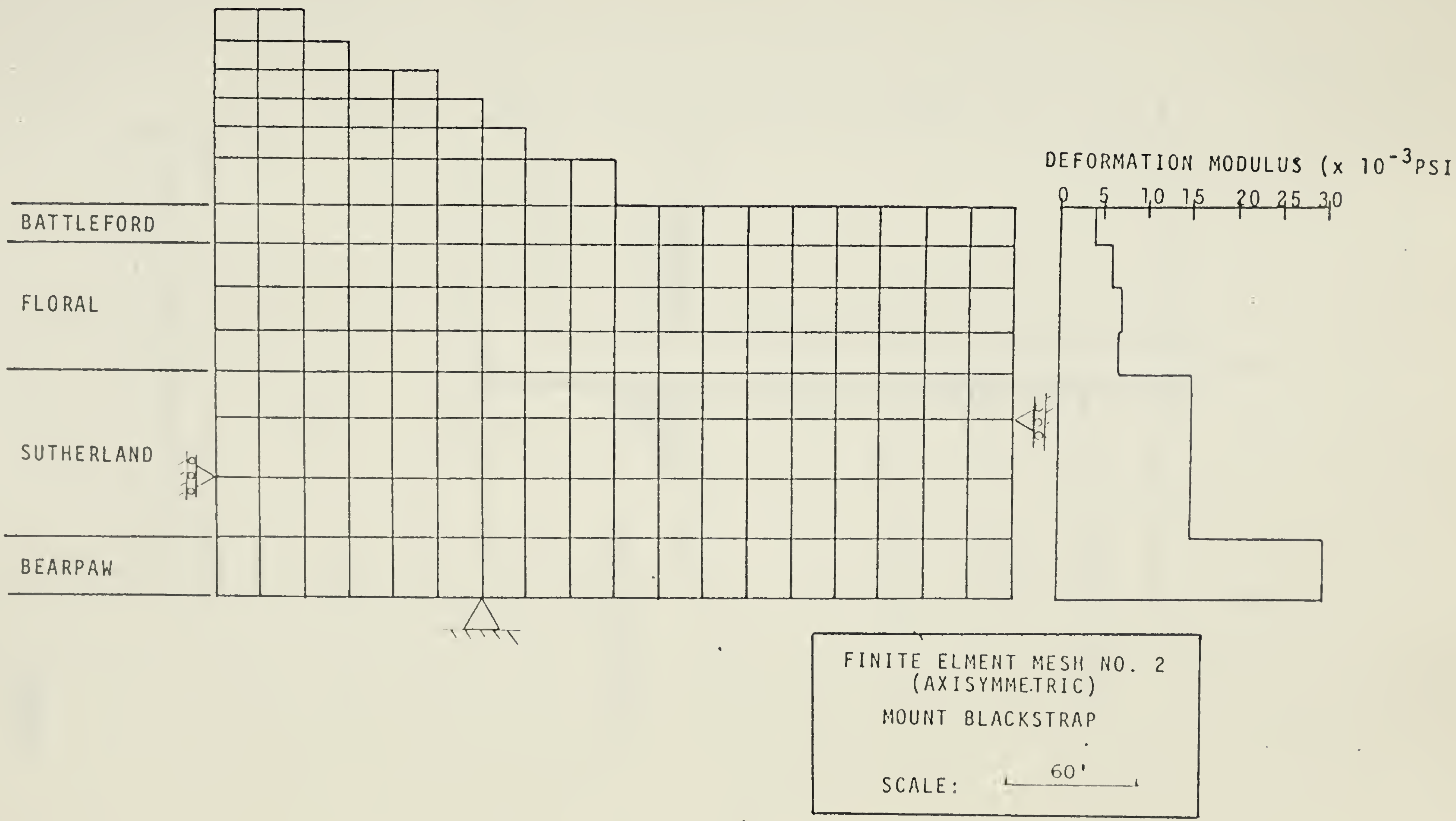
MODULUS PROFILES: MT. BLACKSTRAP
E x 10^-3 (PSI)
0
5
10
15
20
25
30
DEPTH (FT.)
50
100
150
200
BATTLEFORD TILL
FLORAL TILL
SUTHERLAND TILL
BEARPAW SHALE
MODULI USED IN ANALYSIS
LAB. MODULI (FROM HAMILTON & TAO 1972)
RESULTS OF PRESSUREMETER TESTS

FIGURE 6



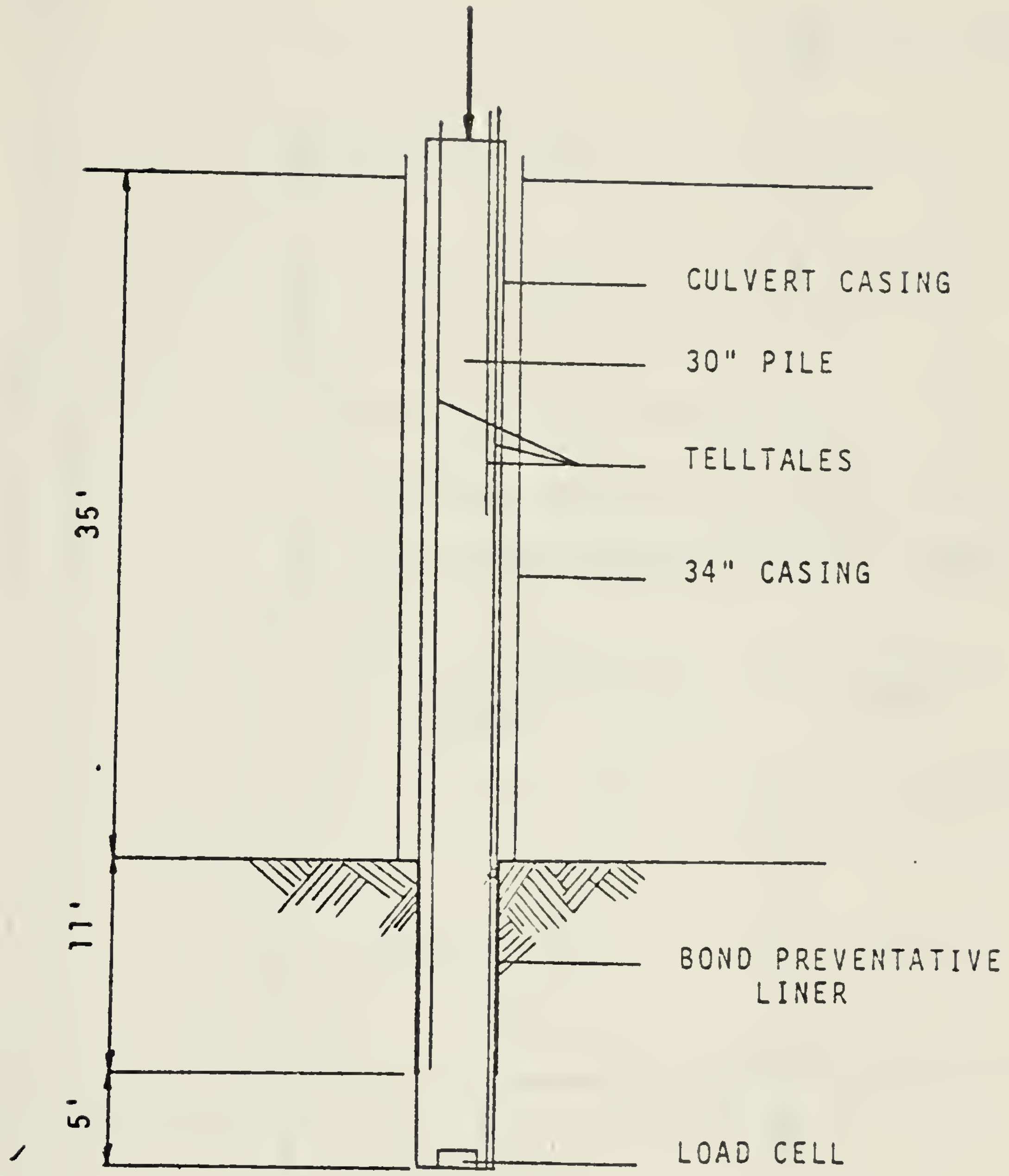
FROM HAMILTON and TAO (1972)

FIGURE 7



BATTLEFORD
FLORAL
SUTHERLAND
BEARPAW
DEFORMATION MODULUS (x 10^{-3}PSI)
0 5 10 15 20 25 30
FINITE ELMENT MESH NO. 2
(AXISYMMETRIC)
MOUNT BLACKSTRAP
SCALE: 60'

FIGURE 8



TEST PILE CONSTRUCTION DETAILS

FIGURE 9

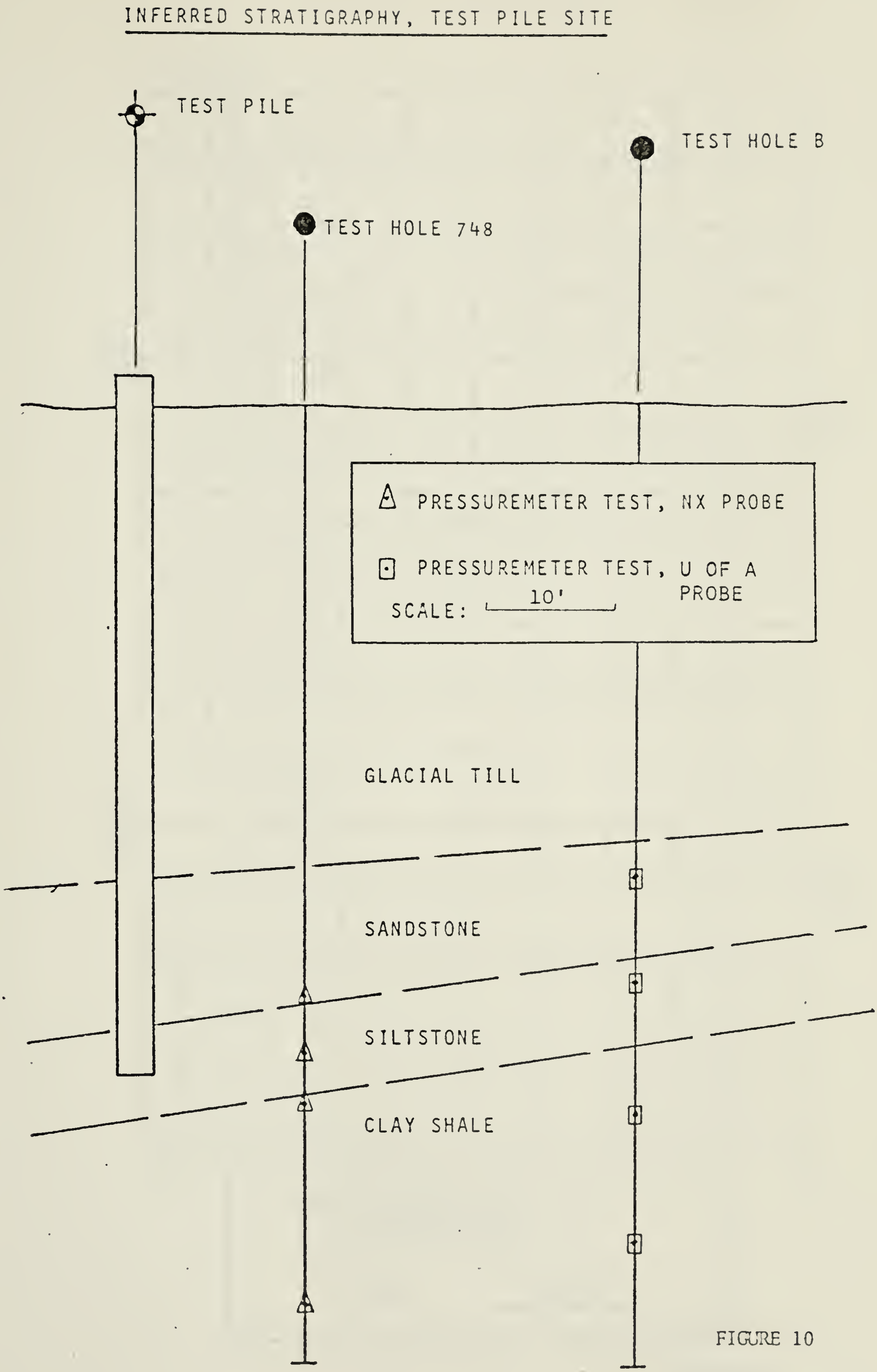

INFERRED STRATIGRAPHY, TEST PILE SITE
TEST PILE
TEST HOLE B
TEST HOLE 748
PRESSUREMETER TEST, NX PROBE
PRESSUREMETER TEST, U OF A PROBE
SCALE: 10'
GLACIAL TILL
SANDSTONE
SILTSTONE
CLAY SHALE

FIGURE 10

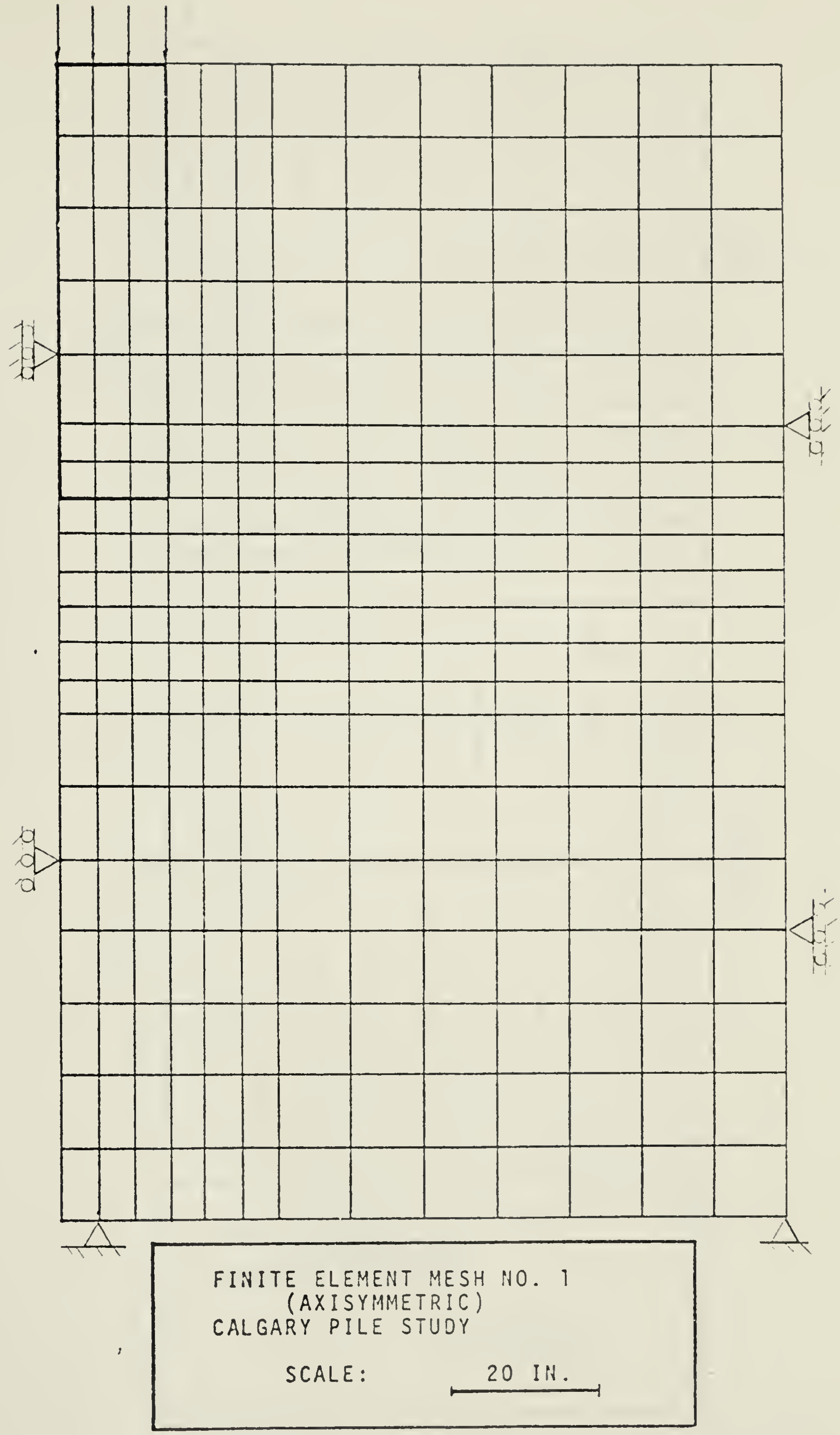
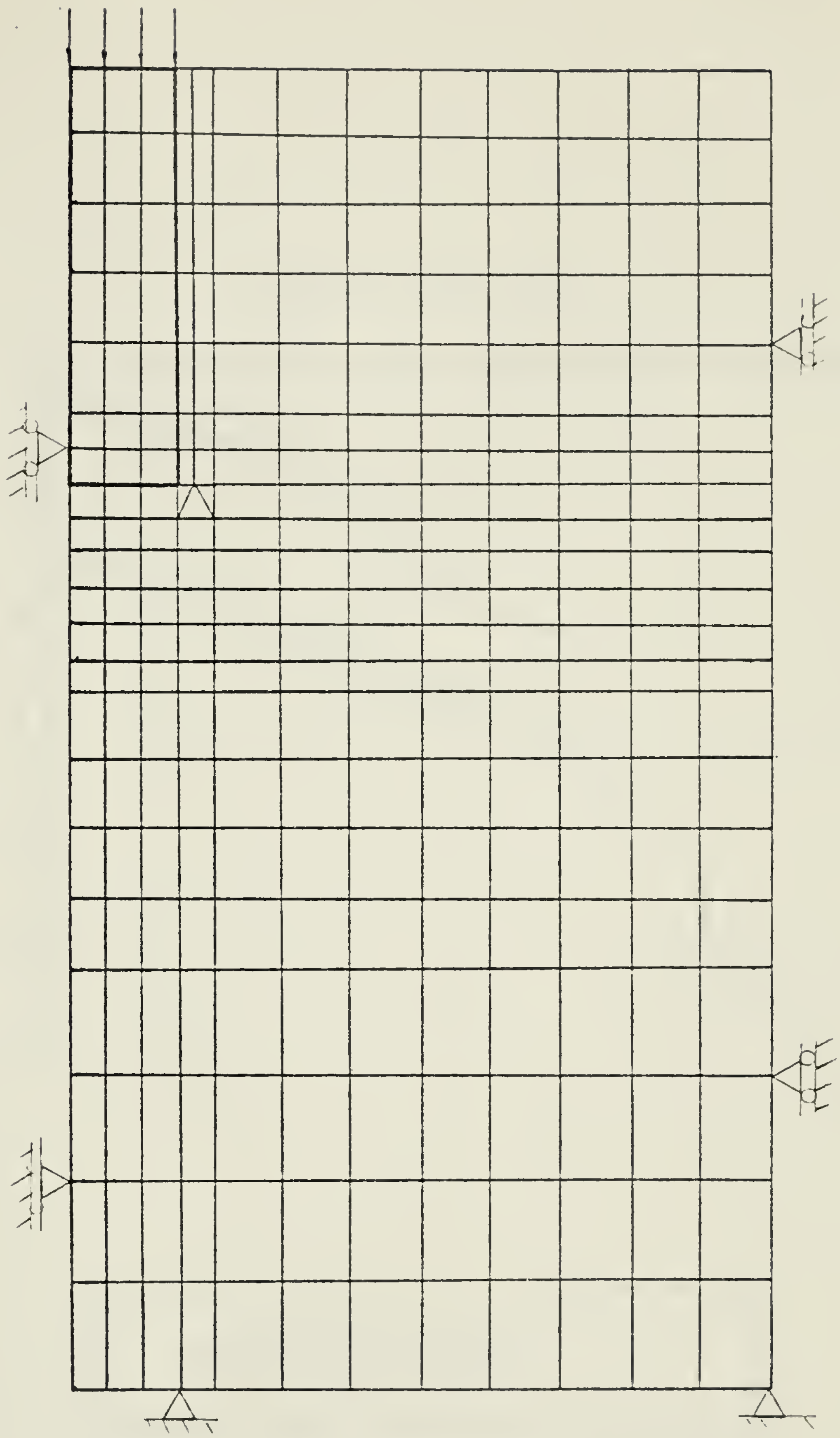

FINITE ELEMENT MESH NO. 1
(AXISYMMETRIC)
CALGARY PILE STUDY
SCALE: 20 IN.

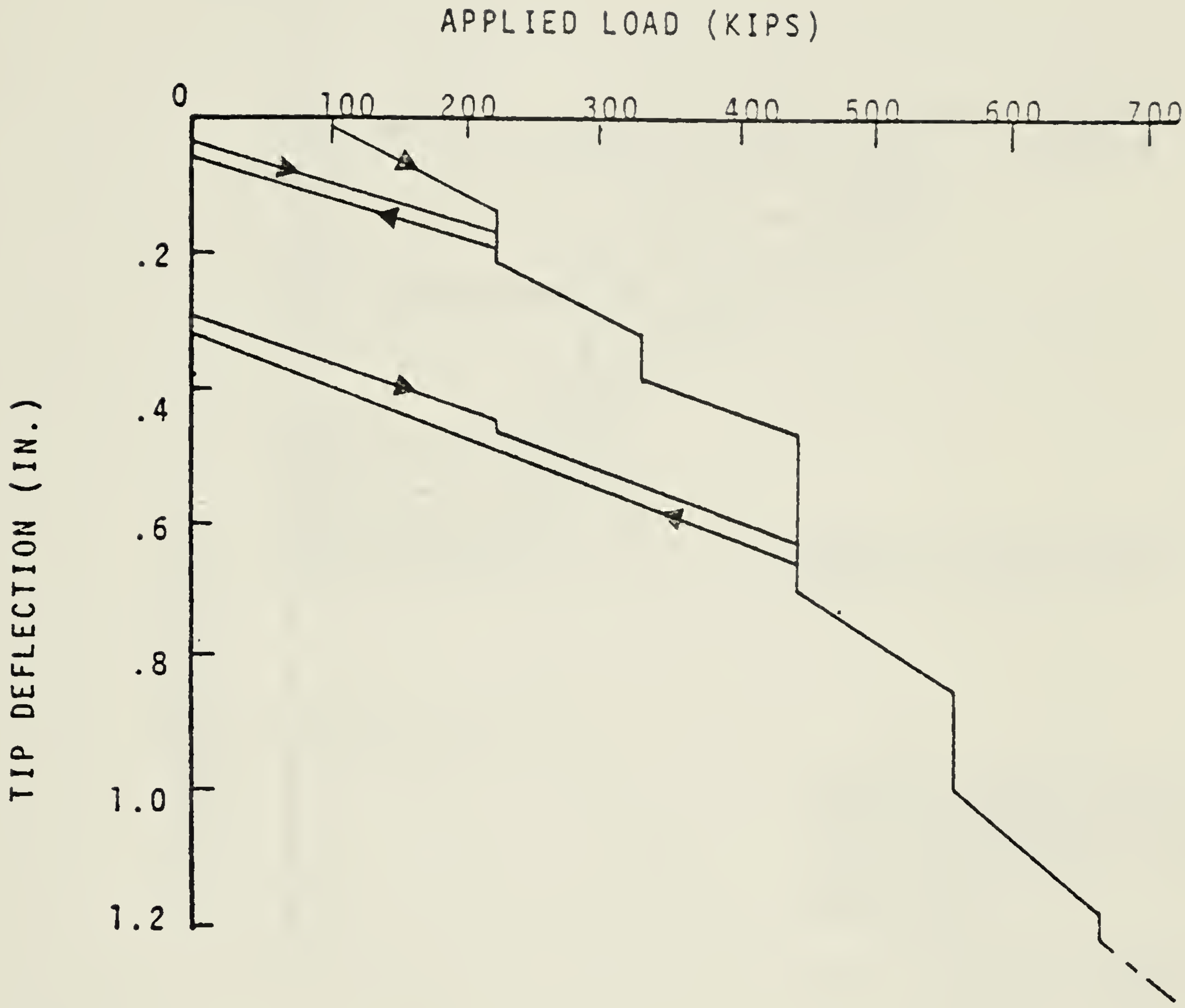
FIGURE 11



FINITE ELEMENT MESH NO. 3
(AXISYMMETRIC)
CALGARY PILE STUDY

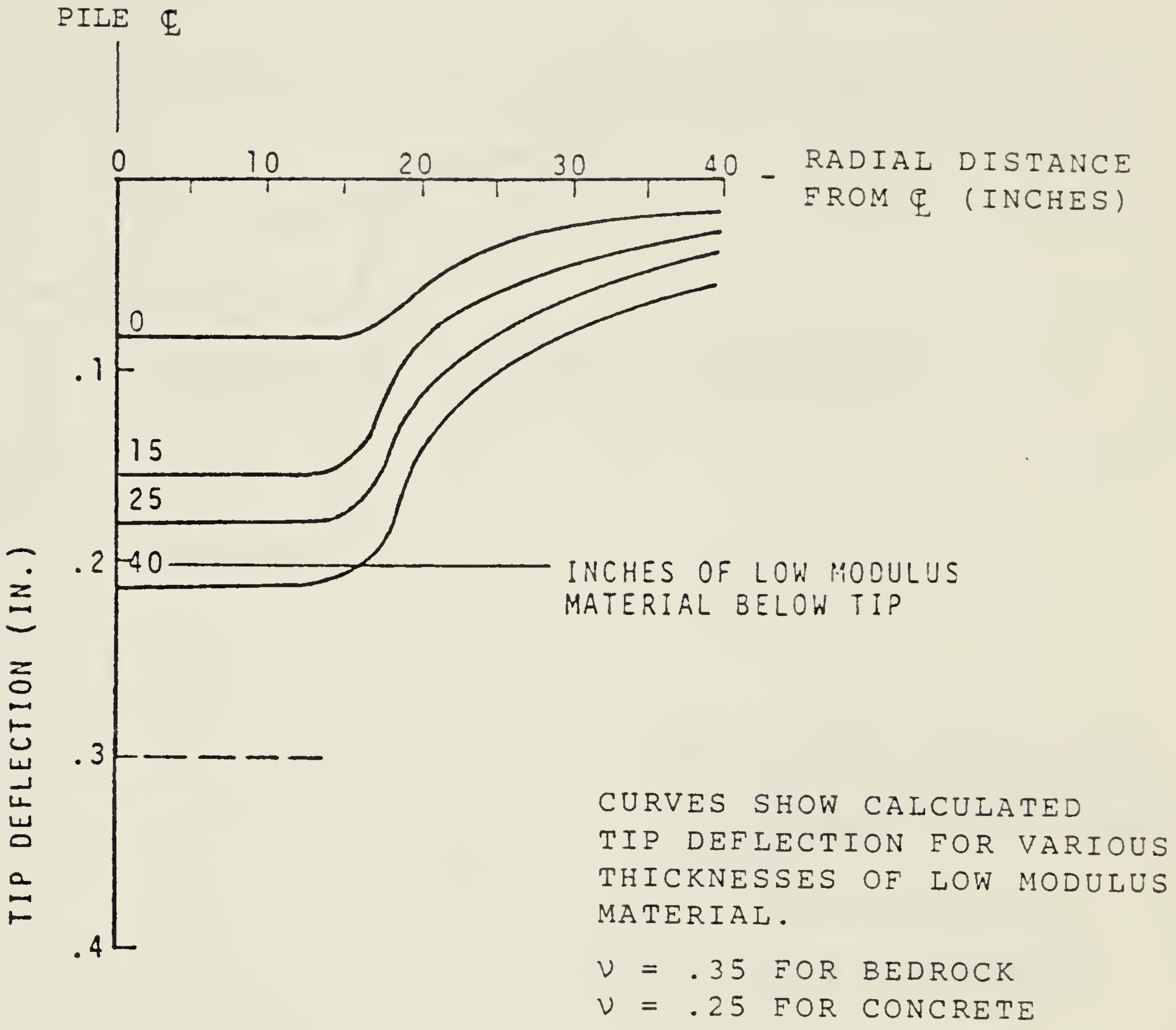
SCALE: 25 IN.

FIGURE 12



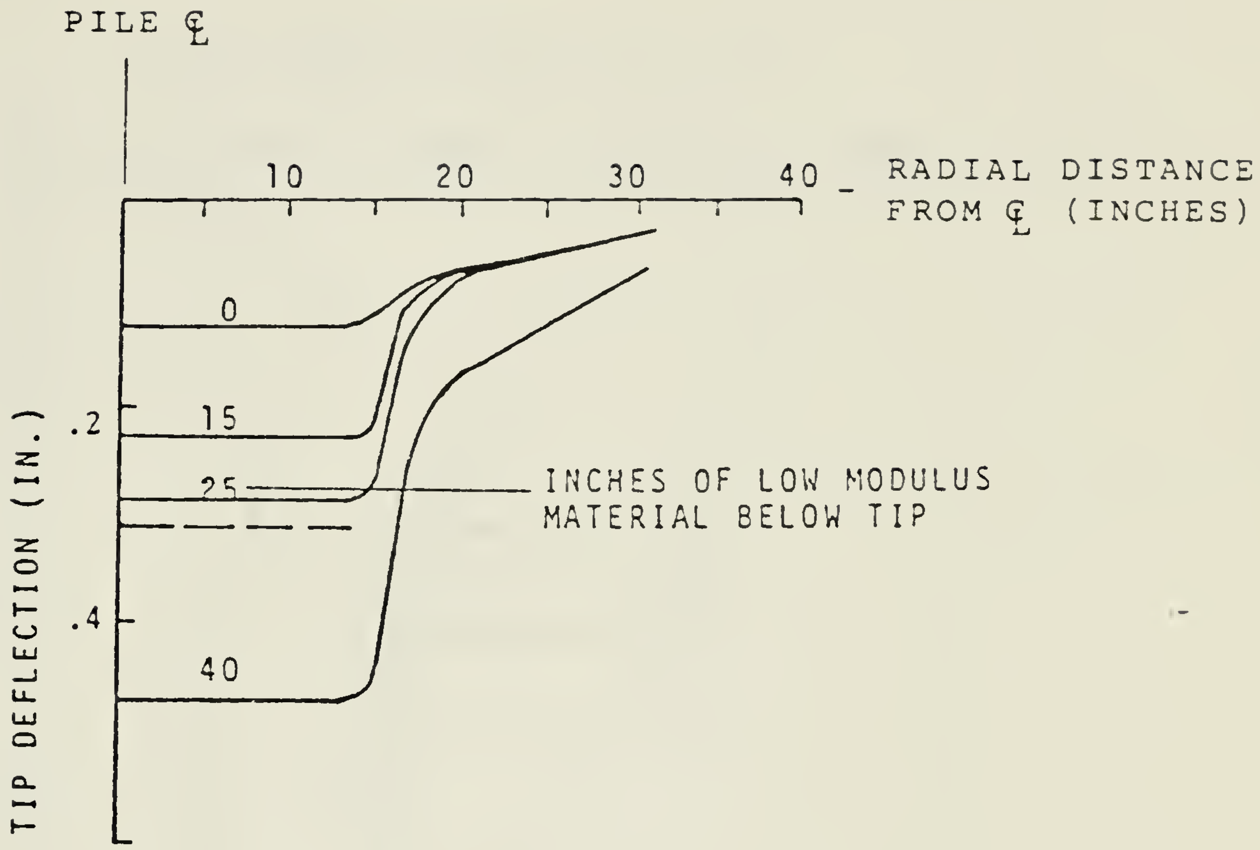
LOAD-DEFLECTION DATA
CALGARY PILE STUDY

FIGURE 13



MESH No. 1 (COMPLETE BONDING)
APPLIED LOAD: 350 KIPS
MEASURED DEFLECTION: 0.30 IN.

FIGURE 14



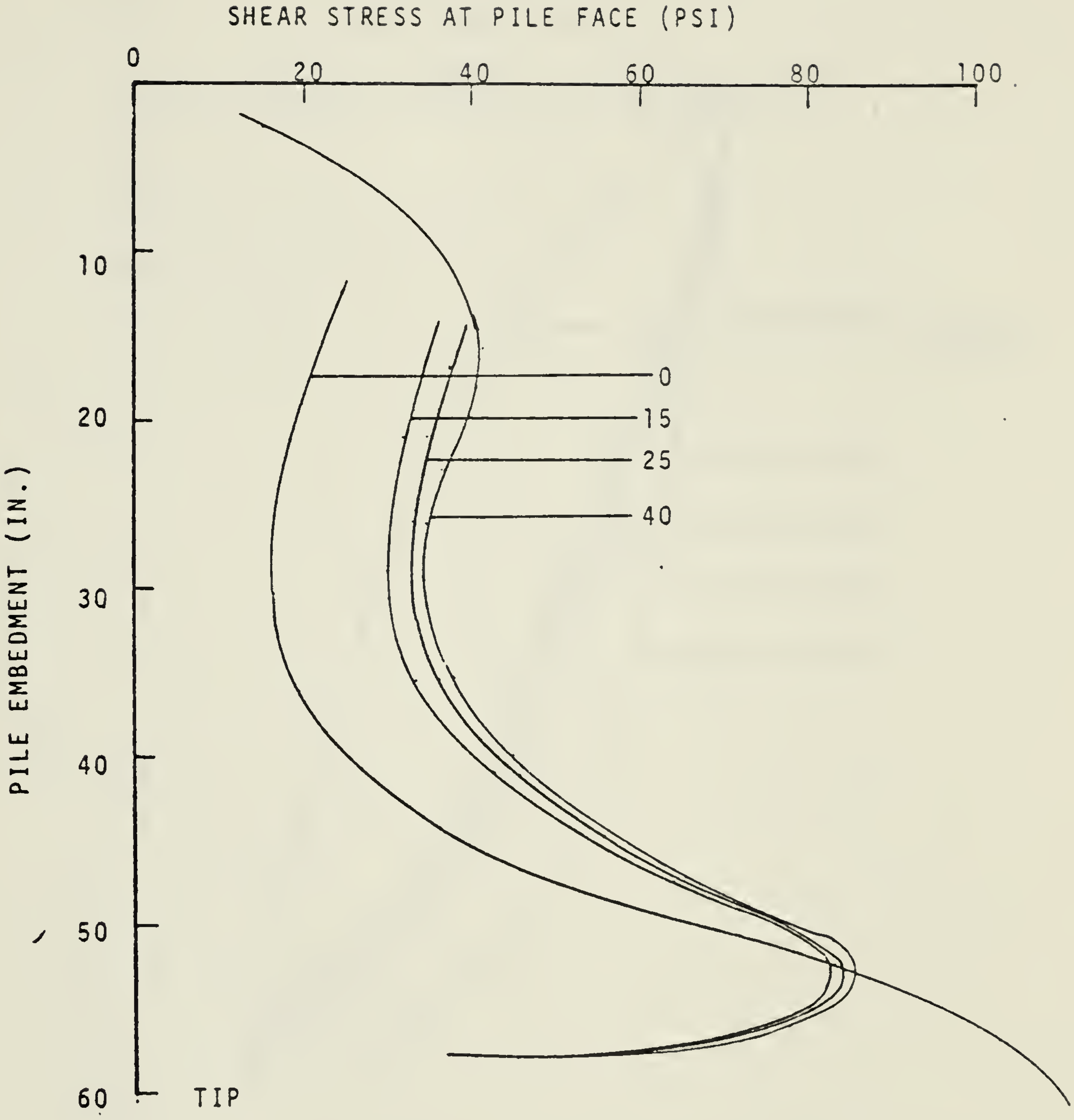
CURVES SHOW CALCULATED TIP DEFLECTION FOR VARIOUS THICKNESSES OF LOW MODULUS MATERIAL.

ν = .35 FOR BEDROCK

ν = .25 FOR CONCRETE

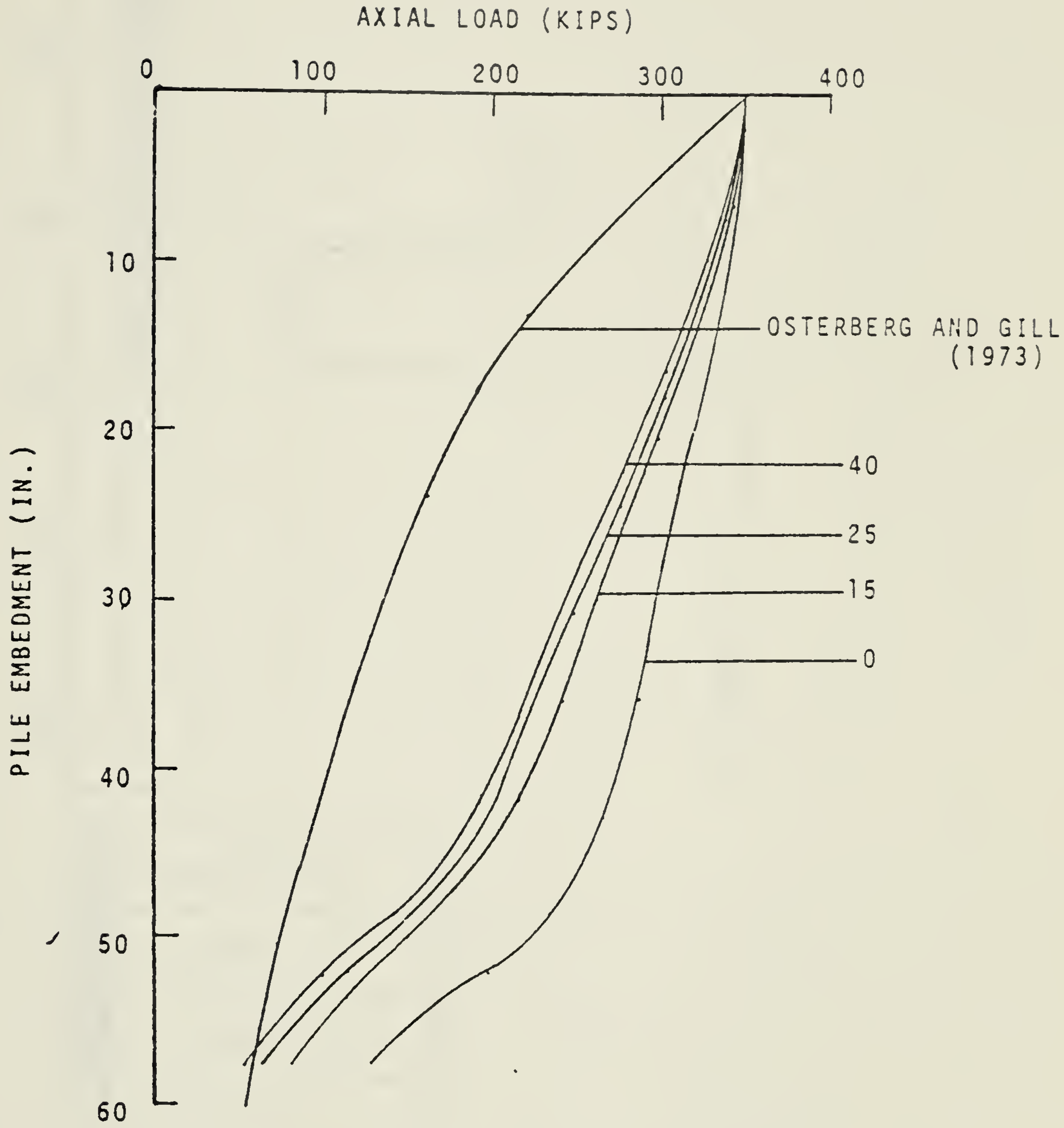
MESH No. 3 (NO BONDING)
APPLIED LOAD: 350 KIPS
MEASURED DEFLECTION: 0.30 IN.

FIGURE 15



SHEAR STRESS DISTRIBUTION OVER EMBEDDED PORTION OF PILE FOR 0, 15, 25 AND 40 INCHES OF LOW MODULUS MATERIAL BELOW PILE TIP

FIGURE 16



AXIAL LOAD DISTRIBUTION FOR 0, 15, 25 AND 40 INCHES OF LOW MODULUS MATERIAL BELOW PILE TIP. COMPARISON CURVE FROM OSTERBERG & GILL (1973) IS FOR 0 INCHES OF LOW MODULUS MATERIAL.

FIGURE 17

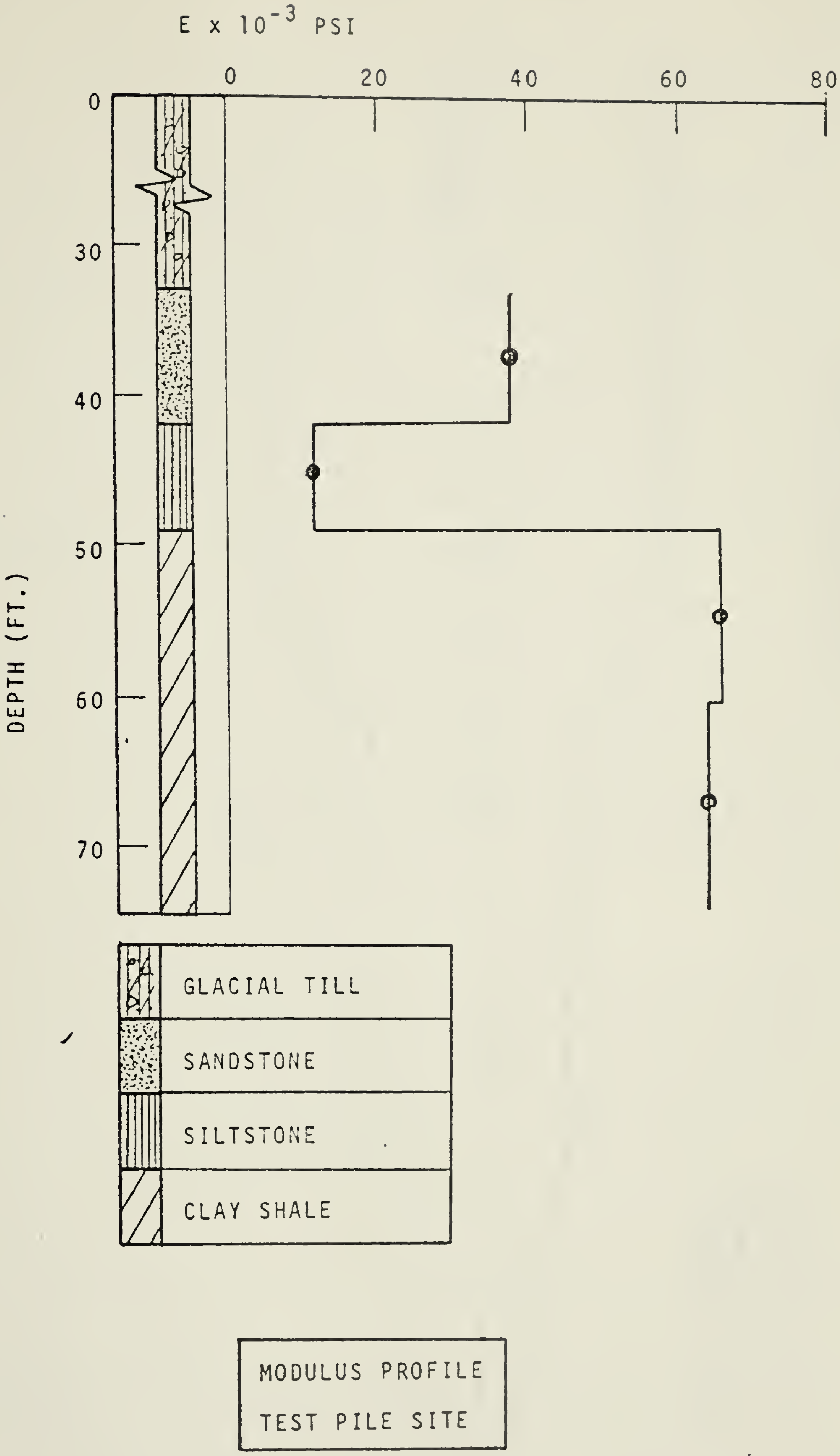
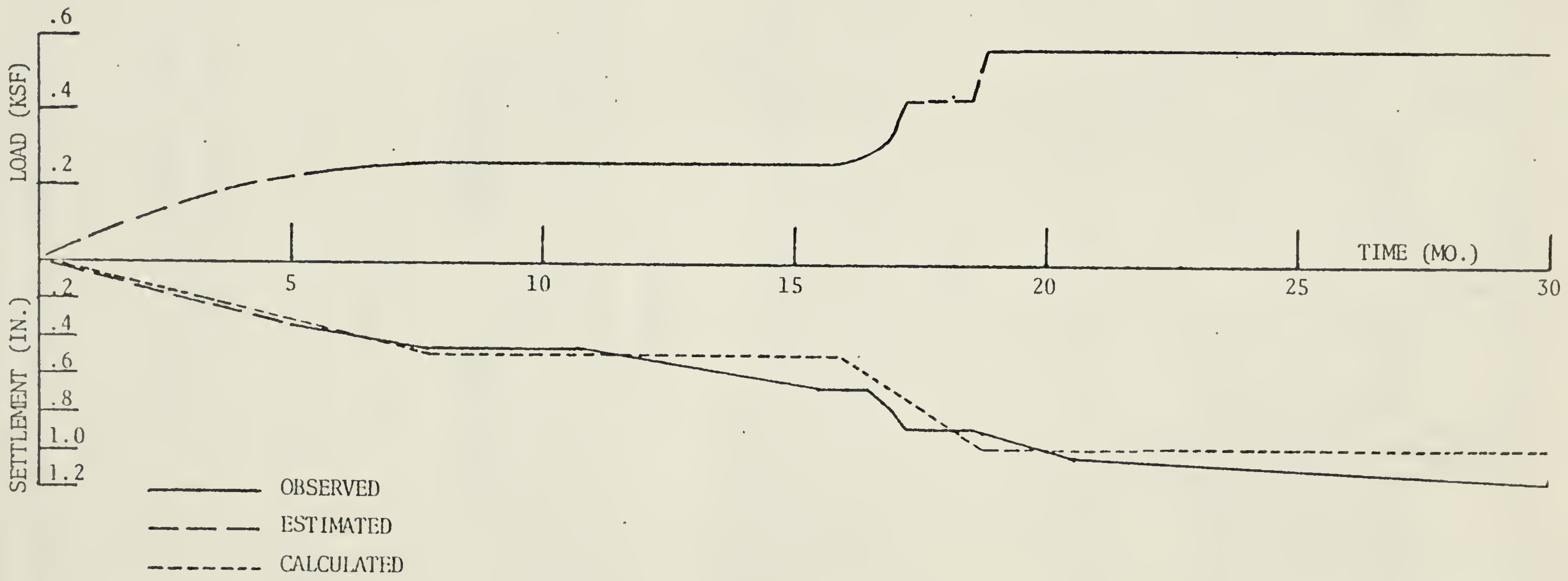

E x 10^{-3} PSI
0
20
40
60
80
0
30
40
50
60
70
DEPTH (FT.)
GLACIAL TILL
SANDSTONE
SILTSTONE
CLAY SHALE
MODULUS PROFILE
TEST PILE SITE

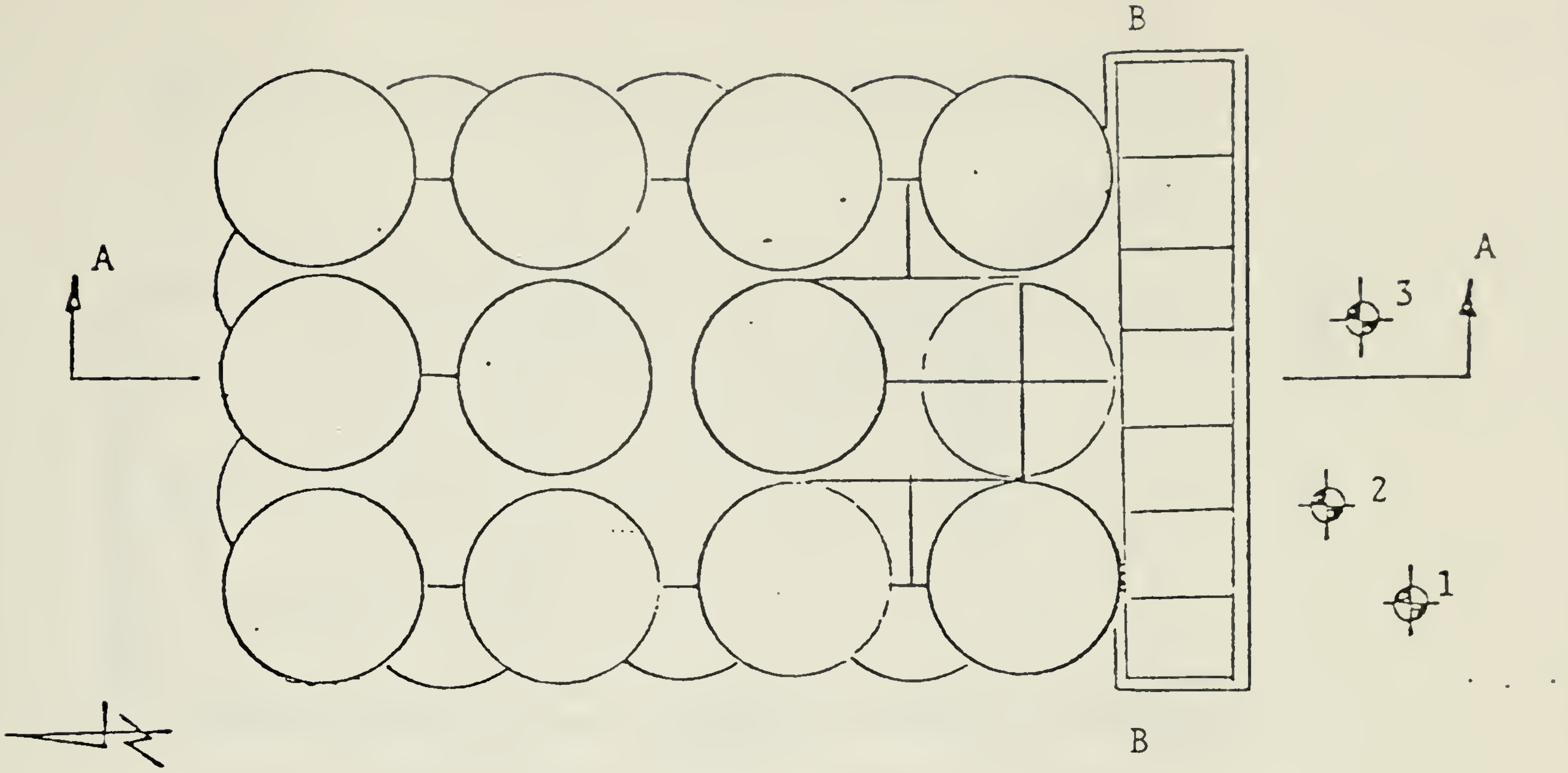
FIGURE 18



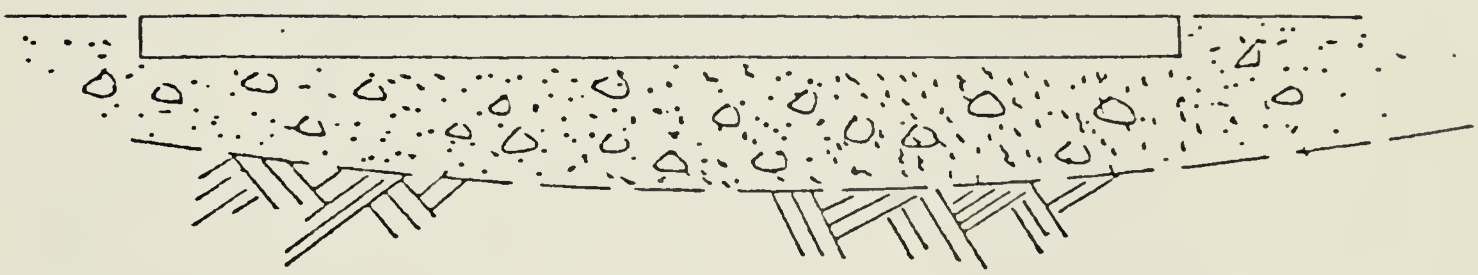
LOAD-TIME-SETTLEMENT DATA
CANADA MALTING STORAGE SILOS
CALGARY, ALBERTA

FIGURE 19

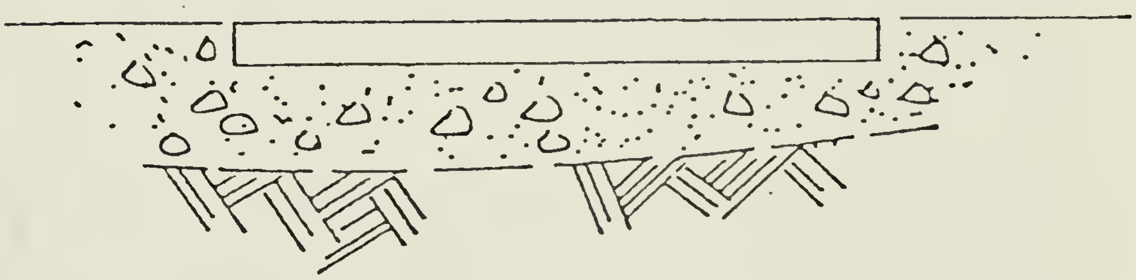
CANADA MALTING CO. STORAGE SILOS



PLAN



SECTION A-A



SECTION B-B

SCALE: 1" = 30'

LEGEND

Gravel- Coarse, dense, clean,
well-rounded particles

Bedrock- Interbedded siltstone, sandstone and
claystone of the Paskapoo Formation

FIGURE 20

FINITE ELEMENT MESH, STRATIGRAPHY AND MODULI USED IN SETTLEMENT ANALYSIS, CANADA MALTING SILOS

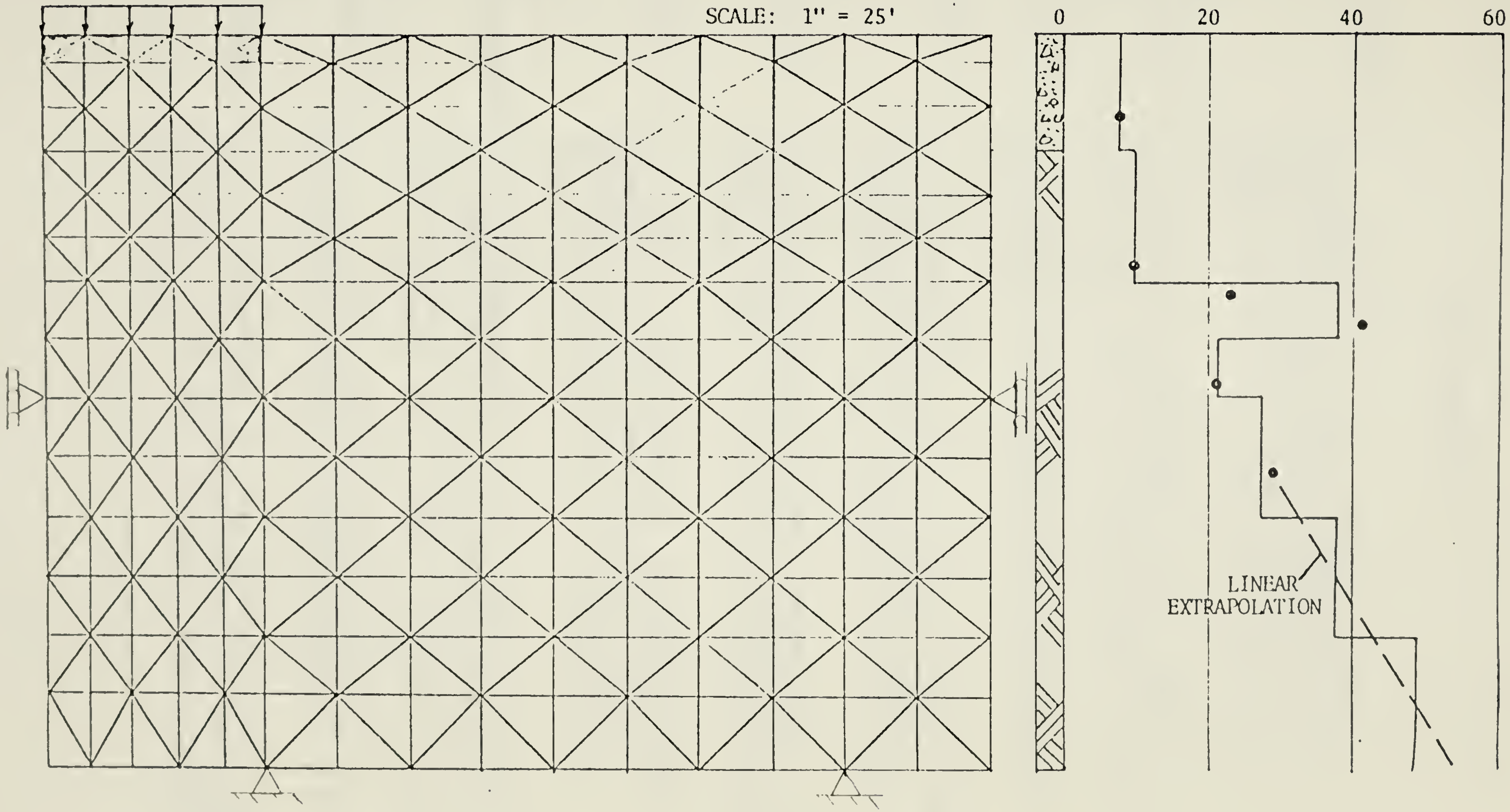


FIGURE 21

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APPENDIX A

INTERPRETATION OF THE PRESSUREMETER TEST AND SAMPLE CALCULATIONS

Several interpretations of the pressuremeter test are possible, these varying mainly with the soil or rock type and whether strength or deformation properties are required.

The immediate portion of bore hole deformation is required in the calculation of a deformation modulus. With the exception of tests in most bedrock materials this quantity is not readily evident and is subject to some judgement. In nearly all cases, a time-dependent relationship occurs for bore hole deformation under an applied pressure, a consequence of discontinuities within the rock mass and the changes in effective stresses within the stressed soil or rock mass.

Pressuremeter tests were performed in glacial till, gravel and soft bedrock in the course of field work for this study. The results of these tests are presented in Appendix B. The calculation of deformation moduli in each case was based on considerations indicated on Figure B1. The immediate portion of bore hole deformation for a given pressure increment was considered to be that volume change which occurred prior to the development of an essentially linear time-volume change relationship. This interpretation leads

to a somewhat smaller deformation modulus than that obtained using other interpretations (Morrison 1972).

Example (1): Calculation of the modulus of deformation of the Floral till at the 45 foot depth at Mount Blackstrap, Saskatchewan. The field record is indicated on Figure B9. From this data the following calculations were made.

FIELD TEST RECORD: Mount Blackstrap

DEPTH: 45 feet

MUD ELEVATION: Ground level

REFERENCE DIAMETER: 4.54 inches

REFERENCE VOLUME READING: 8.2 cm

SEATING VOLUME READING: 43.0 cm

SEATING PRESSURE: 30 psi

FILL TO: 7.4 cm

BLEED TO:

STABLE AT: 7.5 cm

AIR WATER DIFFERENTIAL: 10 psi

Time (Sec.)	Volume Change Readings (pressure, psi)					
	38	50	62	74	88	98
0	7.5	17.9	24.0	30.0	38.2	12.1
5				31.8	40.5	14.0
10	12.0	21.3	26.8	33.1	41.8	15.5
15				33.9	42.5	16.3
20	13.9	22.2	27.6		43.2	17.0
25				34.9	43.8	17.5
30	14.8	22.5	28.0	35.2	44.2	17.9

FIELD TEST RECORD: Mount Blackstrap (continued)

Time (Sec.)	Volume Change Readings (pressure, psi) 38	50	62	74	88	98
35				35.5	44.6	18.3
40	15.4	22.7	28.2	35.7	44.5	18.7
45					45.3	19.0
50		22.8	28.4	36.0	45.5	19.3
60	16.1	22.9	28.5	36.4	45.9	19.8
70		23.0	28.6			
80	16.5	23.1	28.7			
90	16.6		28.9	37.0	47.0	21.0
120	17.0	23.3	29.1	37.5	47.7	21.9
180	17.5	23.7	29.5	38.2	48.9	23.4
240	17.9	24.0	29.9			

REMARKS:
- probe lowered under 10 psi inflation,
- time interval between volume-change measurements from one increment to next = 30 seconds for increments 38-88 psi; 60 sec. for 88-98 psi.
- volume change scales: 50 cc/cm during seating; 5 cc/cm thereafter.

The immediate portion of volume change per pressure increment is indicated by the arrow on each time-volume change curve. Volume change occurring over the final 30 seconds of each of increments 38 to 88 psi is subtracted from the initial portion of the succeeding increment (for

final 60 seconds of increment 88 - 98 psi) to obtain the immediate deformation per pressure increment. These volume changes are plotted versus the applied air pressures. A corrected origin must be used for each time-volume change curve. The reason is that prior to application of the next pressure increment, the water supply to the probe is turned off to establish a starting point for volume change measurement. Often the water level in the sight glass must be readjusted to a more convenient level. For the short period that the water supply is turned off, the probe can continue to expand under the applied air pressure. This volume change is incorporated in the first few seconds of measurement when the water supply is opened at the start of a new pressure increment and must be subtracted from recorded volume change under the higher applied pressure. A convenient time period of either 30 or 60 seconds is usually adopted over which the water supply is turned off and arrangements made for application of a higher pressure.

The borehole volume over the length of the measure cell can be closely estimated from the volume change recorded during seating of the probe. The outside diameter of the probe is measured prior to lowering and this reference diameter and measure cell volume reading recorded. From the volume change required to seat the probe at a pressure close to estimated ground stresses, the actual borehole diameter and volume over the length of the measure cell can be established and is entered as Vo in the equation for deter-

mining the deformation modulus. The equation commonly used is

$$E = \frac{2(1+\nu)\ \Delta p}{\Delta v/Vo}$$

where E = deformation modulus (psi)

ν = Poisson's ratio

$\Delta v/\Delta p$ = slope of linear portion of the pressure-volume change curve

Vo = original borehole volume over the length of the measure cell at an applied pressure roughly equal to estimated original ground stresses.

For the case of this example, the following quantities are calculated:

1) $\Delta v = \frac{\pi}{4} (D_2{}^2 - D_1{}^2)\ (L_c)\ (16.4)$

where

Δv = seating volume change

$D_2{}^2$ = diameter of measure cell after seating of probe

$D_1{}^2$ = reference diameter of measure cell prior to lowering of probe (probe O.D. minus wall thickness of external membrane).

L_c = length of measure cell in inches (13 inches)

16.4 = conversion factor, cubic inches to cubic centimeters

Rearranging, $D_2^{\,2} = (\frac{\Delta V}{214} + .785\ D_1^{\,2})\ 1.27$

and $\Delta V = (43.0 - 8.2) \times 50$

$= 1740$ cc

$D_1 = 4.54 - .8$

$= 3.74$ in.

Solving, $D_2 = 4.93$ inches and

$Vo = \frac{\pi}{4}\ (4.93 + .8)^2\ (16.4)\ (13)$

$= 5500$ cc

2) $$E = \frac{2(1 + \nu)\ \Delta p}{\Delta v/Vo}$$

where

- from the pressure-volume change curve of Figure A,

$$\frac{\Delta v}{\Delta p} = \frac{25.5}{100}$$

- Vo = 5500 cc

Assuming $\nu = 0.40$, $E = 6050$ psi

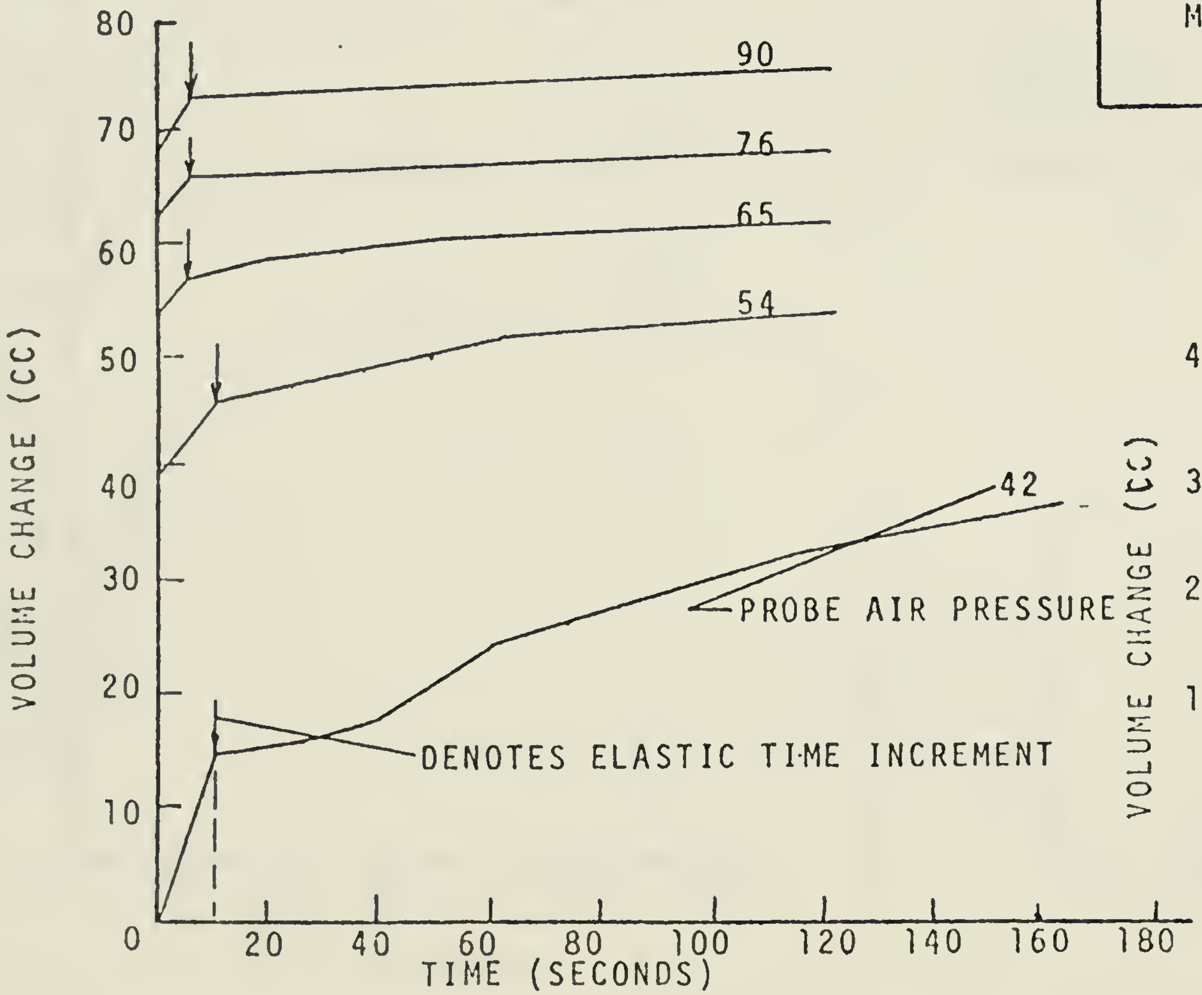
APPENDIX B

RESULTS OF FIELD TEST PROGRAMS

FIELD TEST RESULTS
TEST PILE SITE
CALGARY AIRPORT

DEPTH: 38'
MATERIAL: SANDSTONE
MODULUS: 36,000 PSI

TIME - VOLUME CHANGE CURVES



PRESSURE - VOLUME CHANGE CURVE

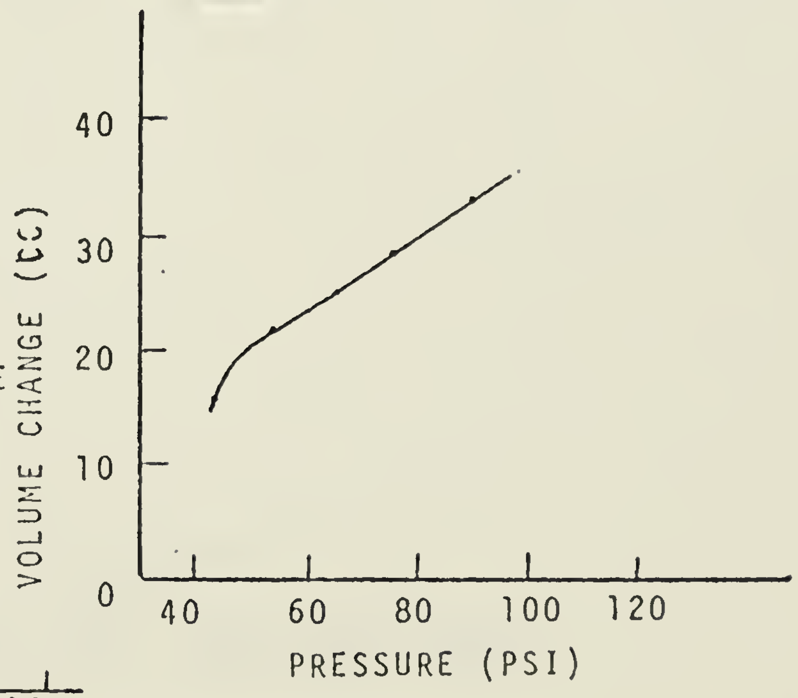


FIGURE B1

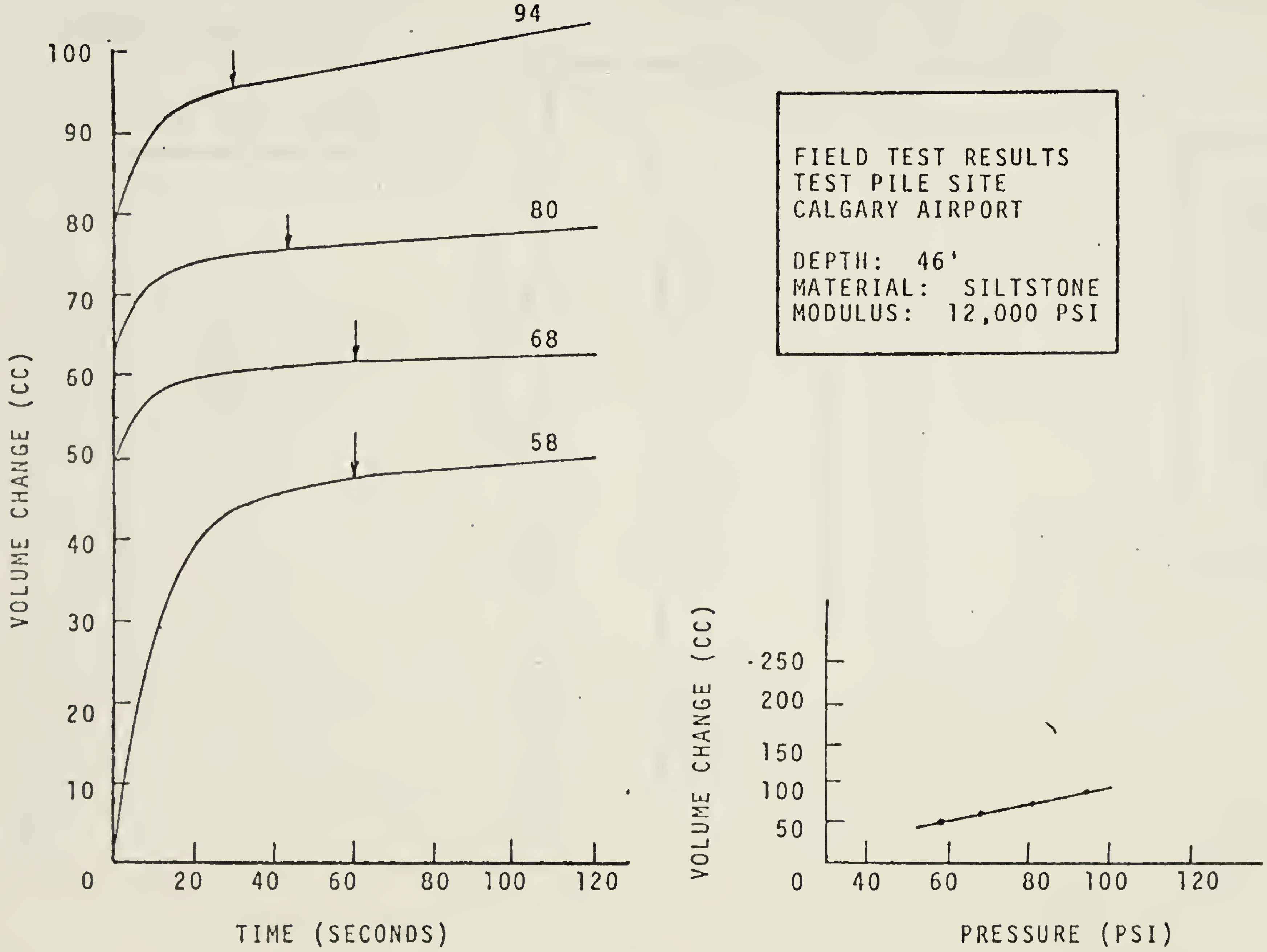

FIELD TEST RESULTS
TEST PILE SITE
CALGARY AIRPORT
DEPTH: 46'
MATERIAL: SILTSTONE
MODULUS: 12,000 PSI
94
80
68
58
VOLUME CHANGE (CC)
100
90
80
70
60
50
40
30
20
10
0
20
40
60
80
100
120
TIME (SECONDS)
VOLUME CHANGE (CC)
250
200
150
100
50
0
40
60
80
100
120
PRESSURE (PSI)

FIGURE B2

FIELD TEST RESULTS
TEST PILE SITE
CALGARY AIRPORT

DEPTH: 56'
MATERIAL: CLAYSHALE
MODULUS: 66,000 PSI

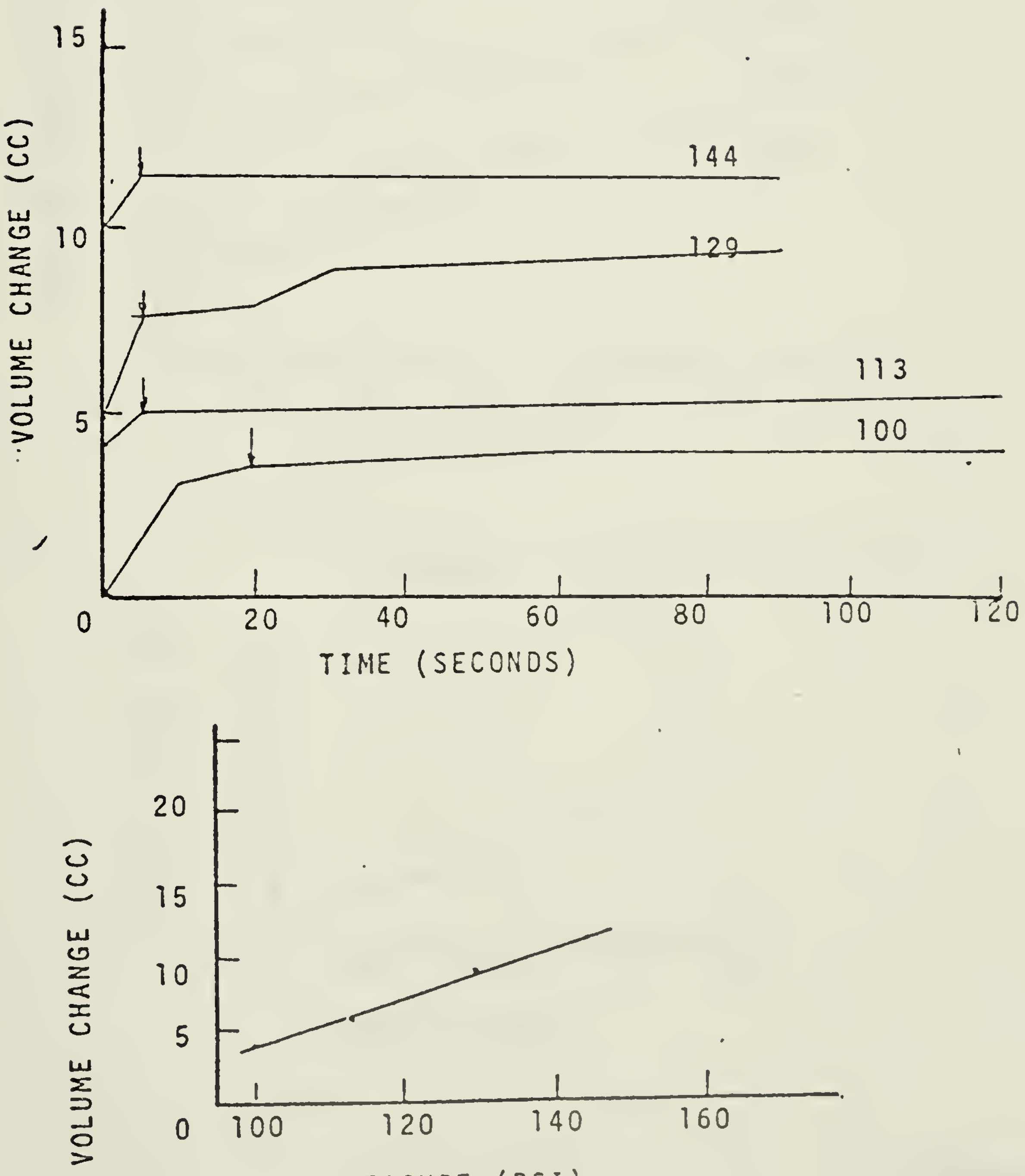
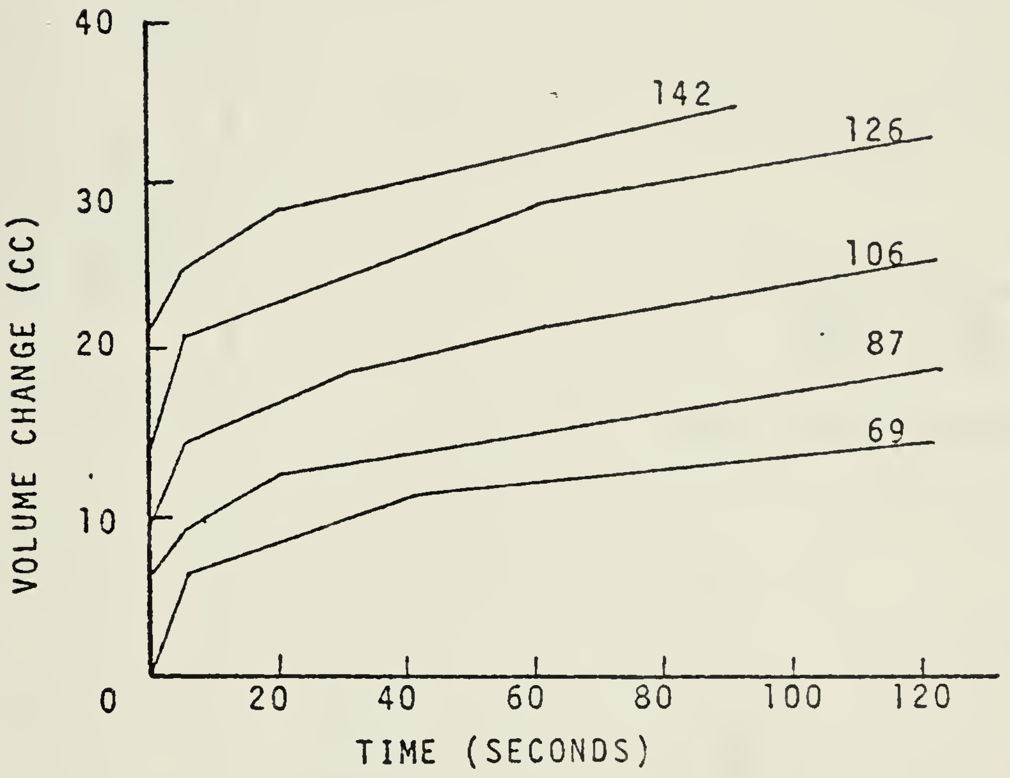


FIGURE B3

FIELD TEST RESULTS
TEST PILE SITE
CALGARY AIRPORT

DEPTH: 66'
MATERIAL: CLAYSHALE
MODULUS: 64,000 PSI



PRESSURE - VOLUME CHANGE CURVE

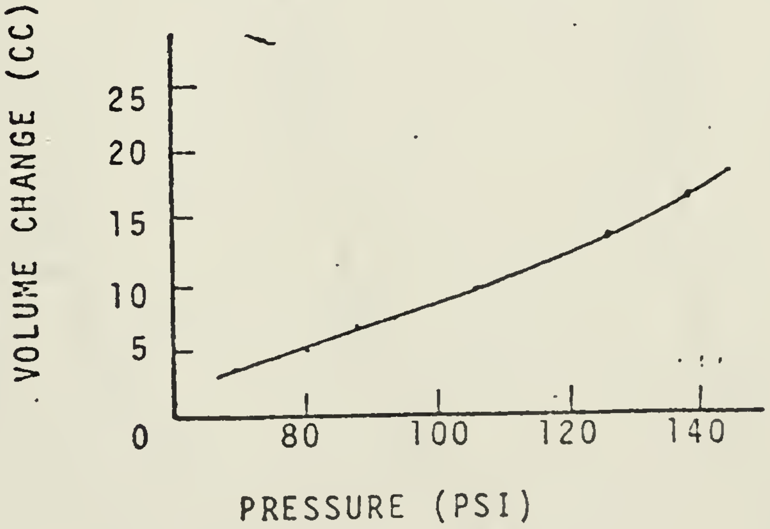
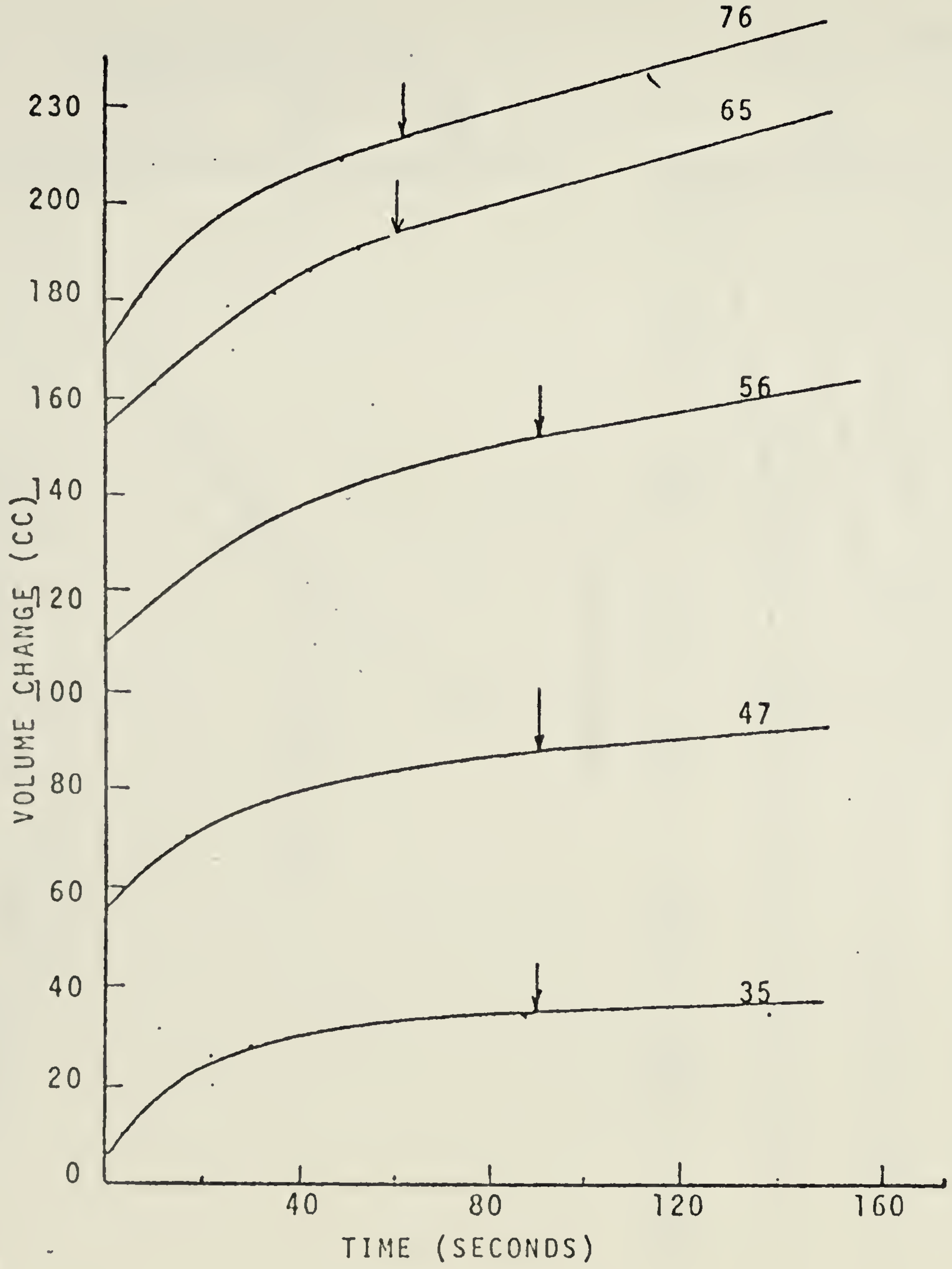


FIGURE B4



FIELD TEST RESULTS
MOUNT BLACKSTRAP

DEPTH: 9'
MATERIAL: TILL (BATTLEFORD)
MODULUS: 830 PSI

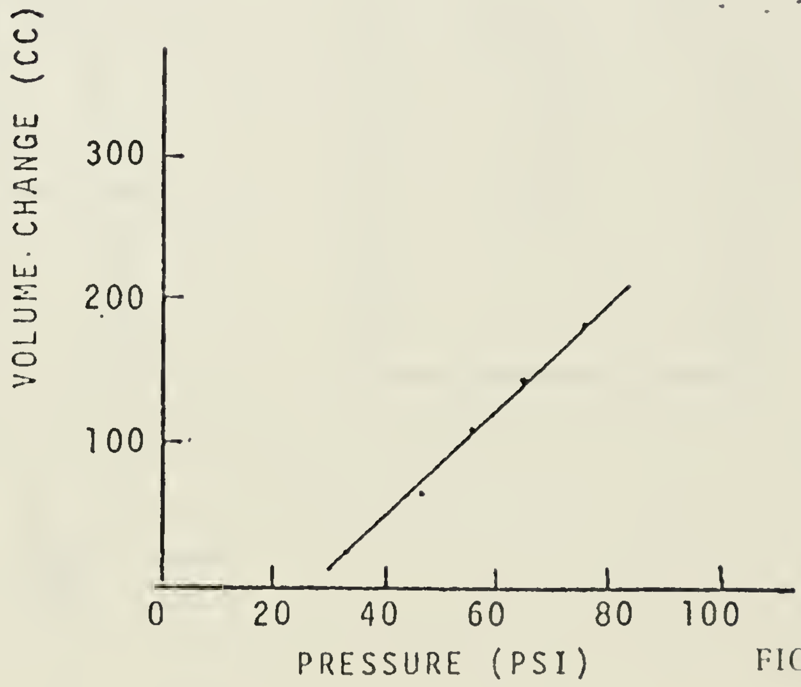
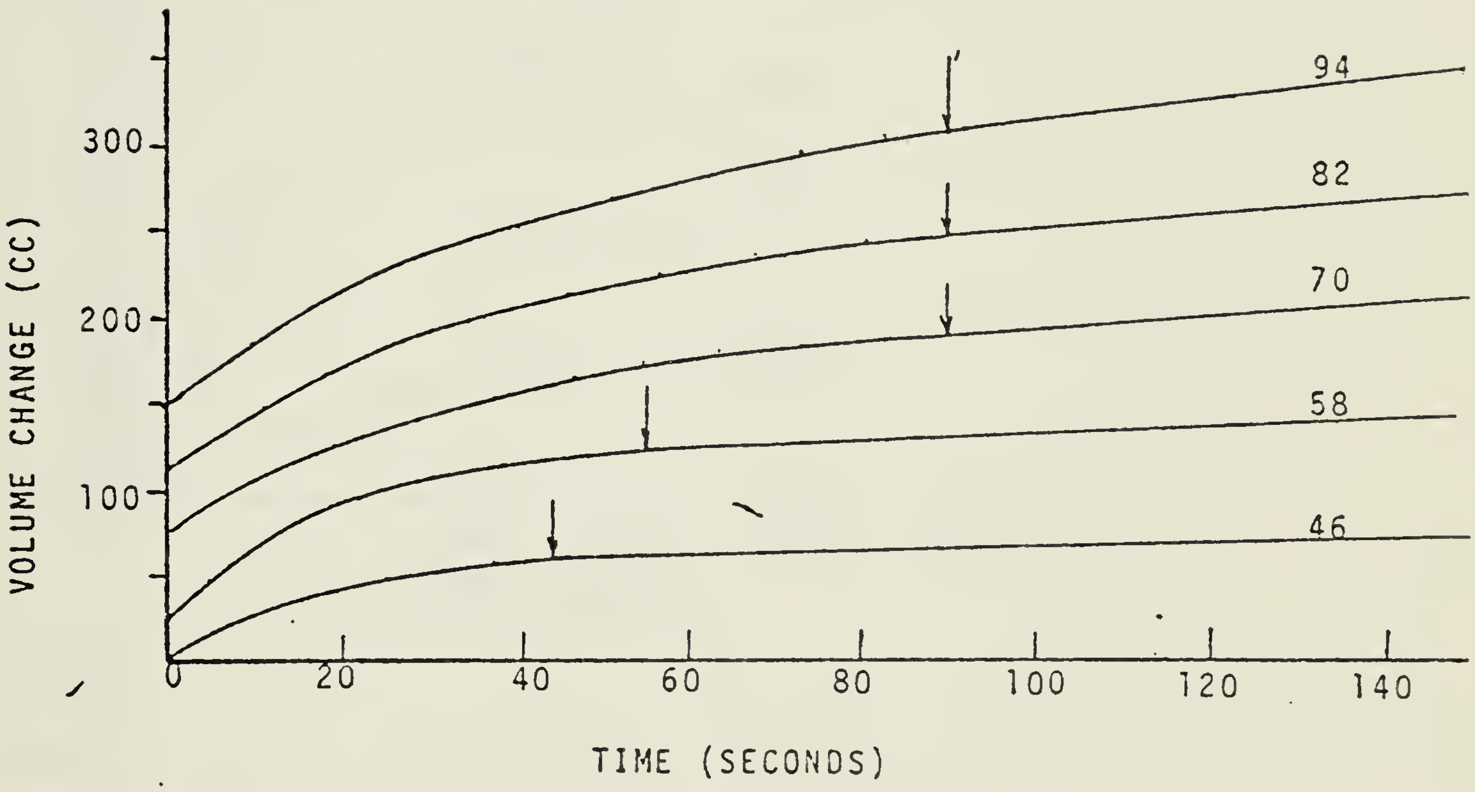


FIGURE B5

FIELD TEST RESULTS
MOUNT BLACKSTRAP

DEPTH: 17'
MATERIAL: TILL (BATTLEFORD)
MODULUS: 4050 PSI



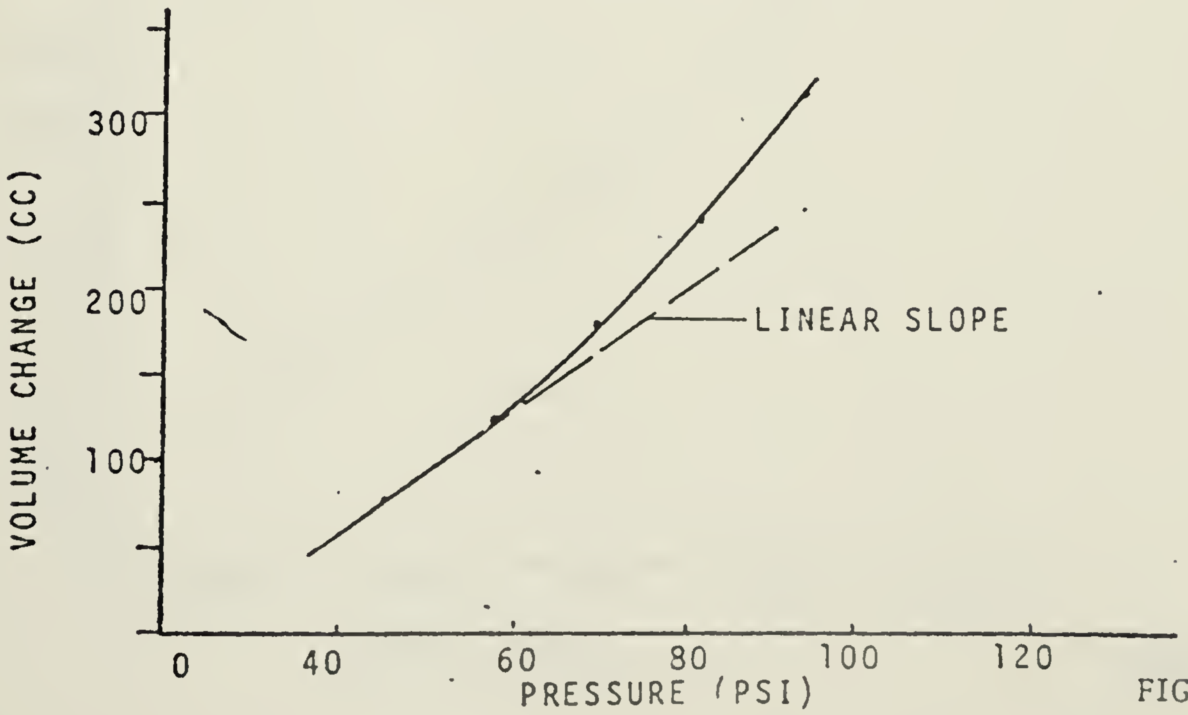
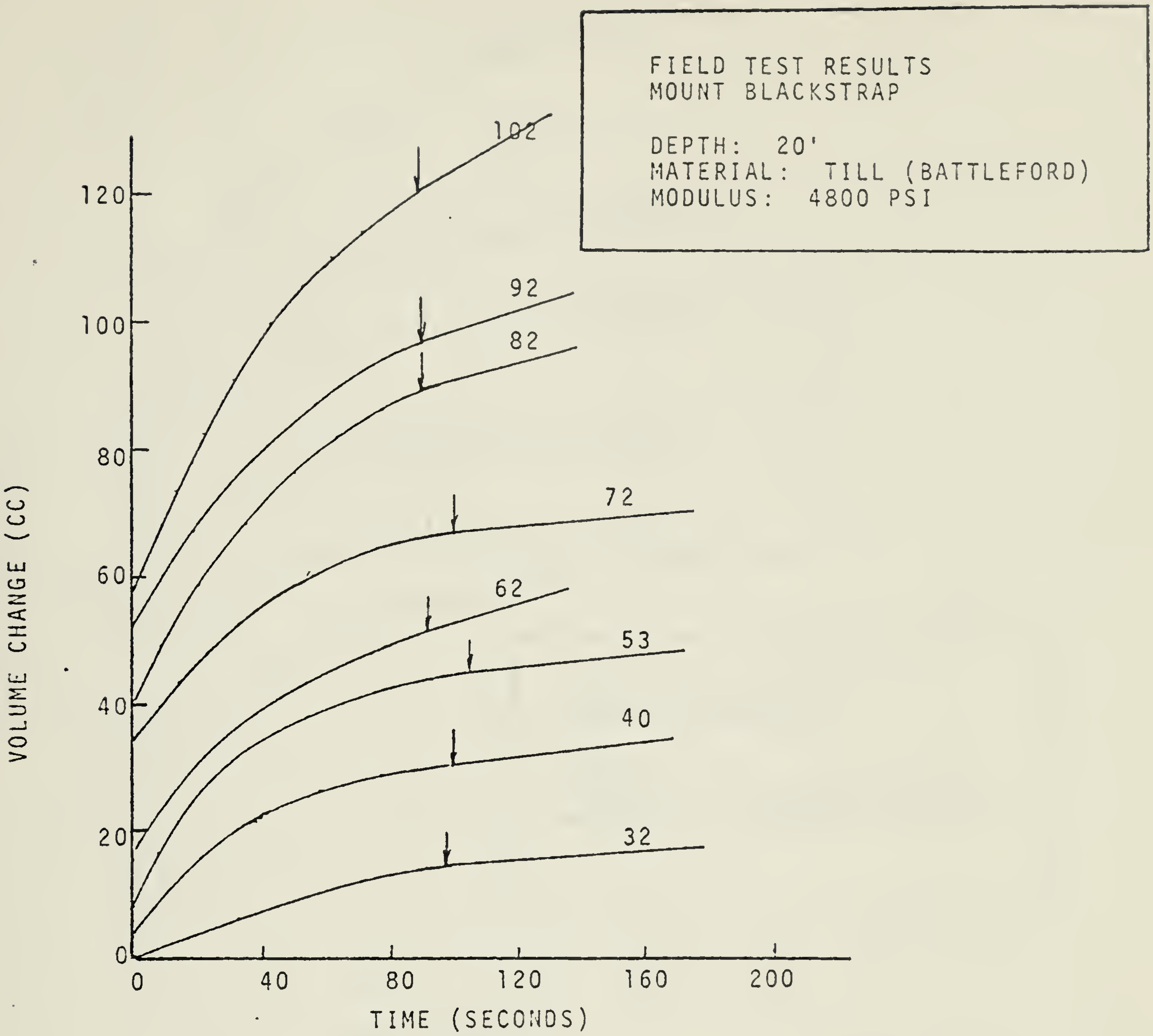


FIGURE B6


FIELD TEST RESULTS
MOUNT BLACKSTRAP
DEPTH: 20'
MATERIAL: TILL (BATTLEFORD)
MODULUS: 4800 PSI
102
92
82
72
62
53
40
32
VOLUME CHANGE (CC)
TIME (SECONDS)
0
40
80
120
160
200
20
60
100

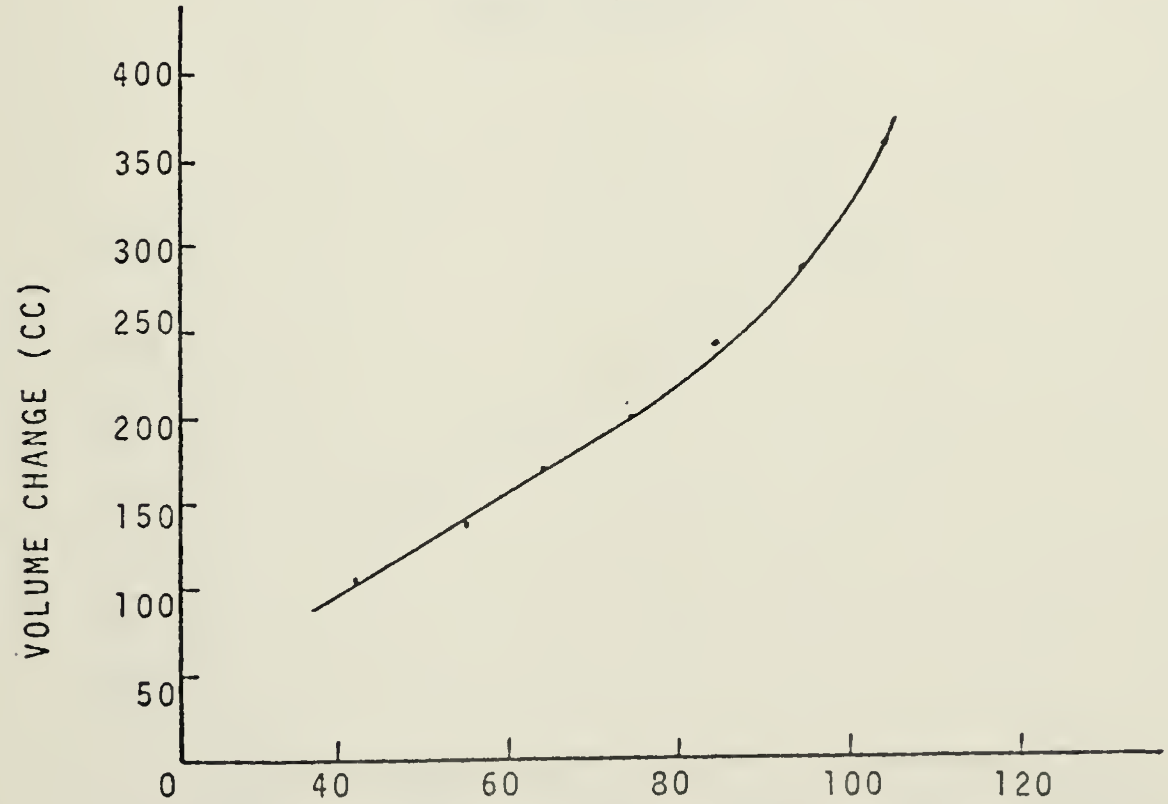
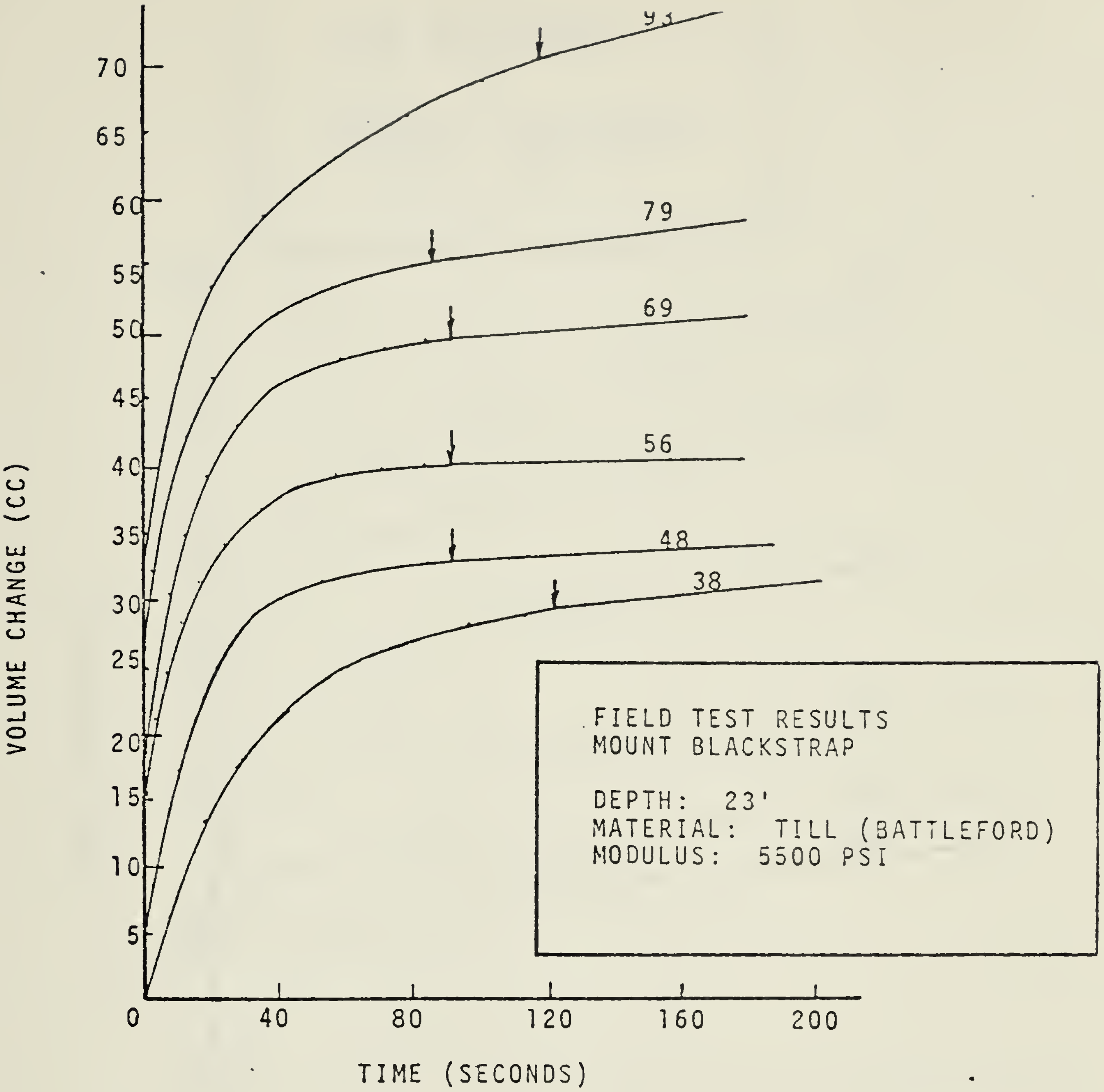

VOLUME CHANGE (CC)
PRESSURE (PSI)
50
100
150
200
250
300
350
400
0
40
60
80
100
120

FIGURE B7


VOLUME CHANGE (CC)
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
40
80
120
160
200
TIME (SECONDS)
79
69
56
48
38
FIELD TEST RESULTS
MOUNT BLACKSTRAP
DEPTH: 23'
MATERIAL: TILL (BATTLEFORD)
MODULUS: 5500 PSI

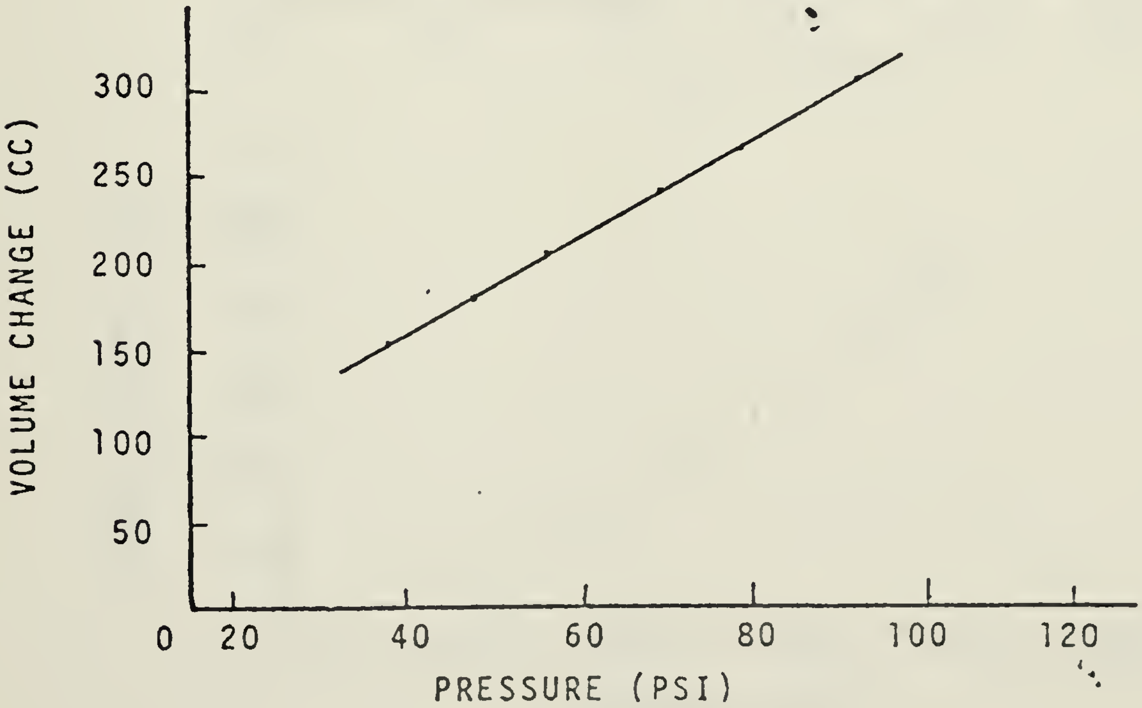
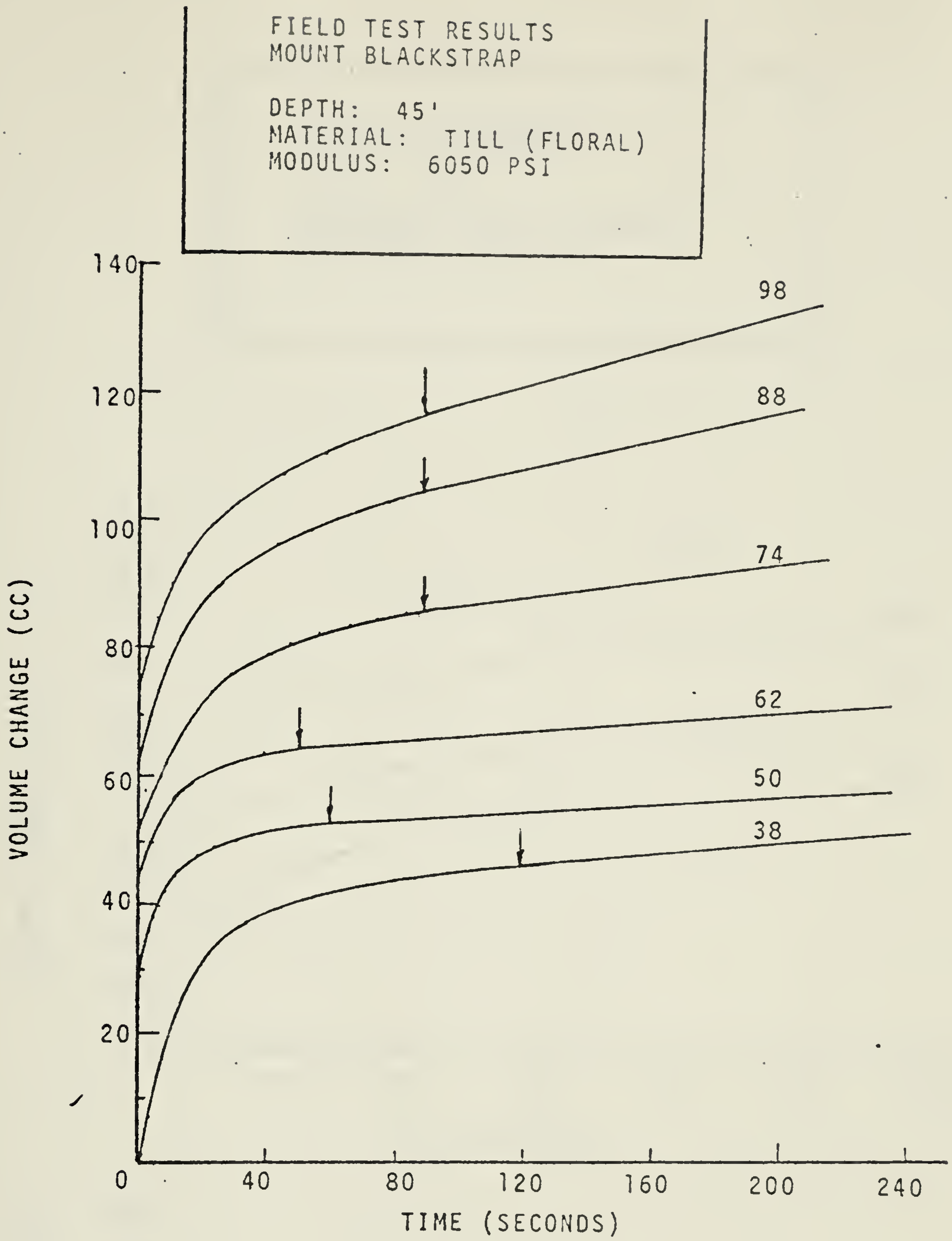

VOLUME CHANGE (CC)
300
250
200
150
100
50
0
20
40
60
80
100
120
PRESSURE (PSI)

FIGURE B8


FIELD TEST RESULTS
MOUNT BLACKSTRAP
DEPTH: 45'
MATERIAL: TILL (FLORAL)
MODULUS: 6050 PSI
VOLUME CHANGE (CC)
140
120
100
80
60
40
20
0
40
80
120
160
200
240
TIME (SECONDS)
98
88
74
62
50
38

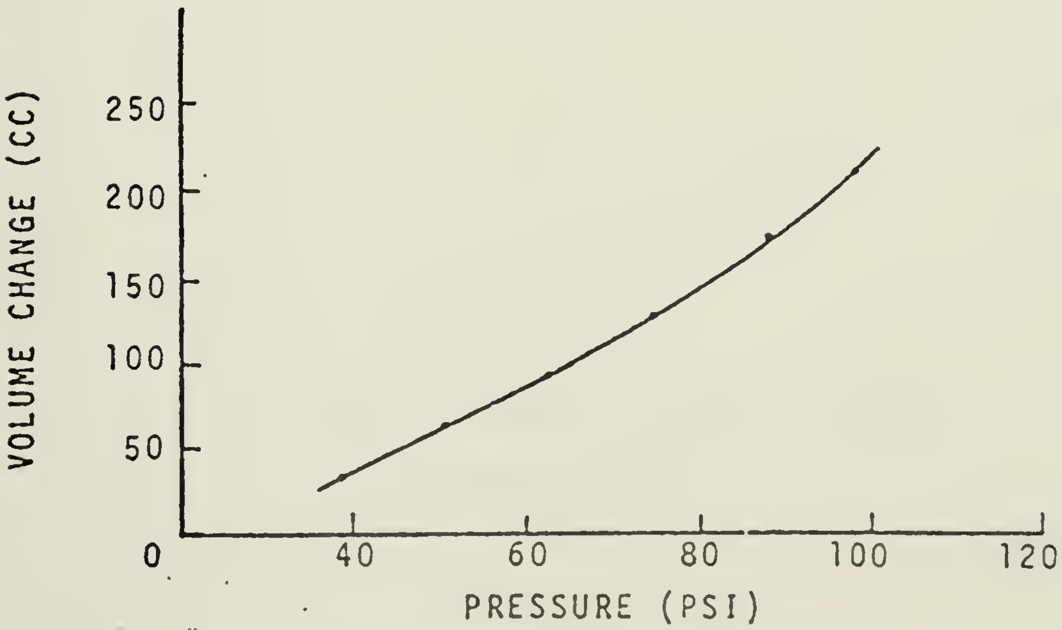
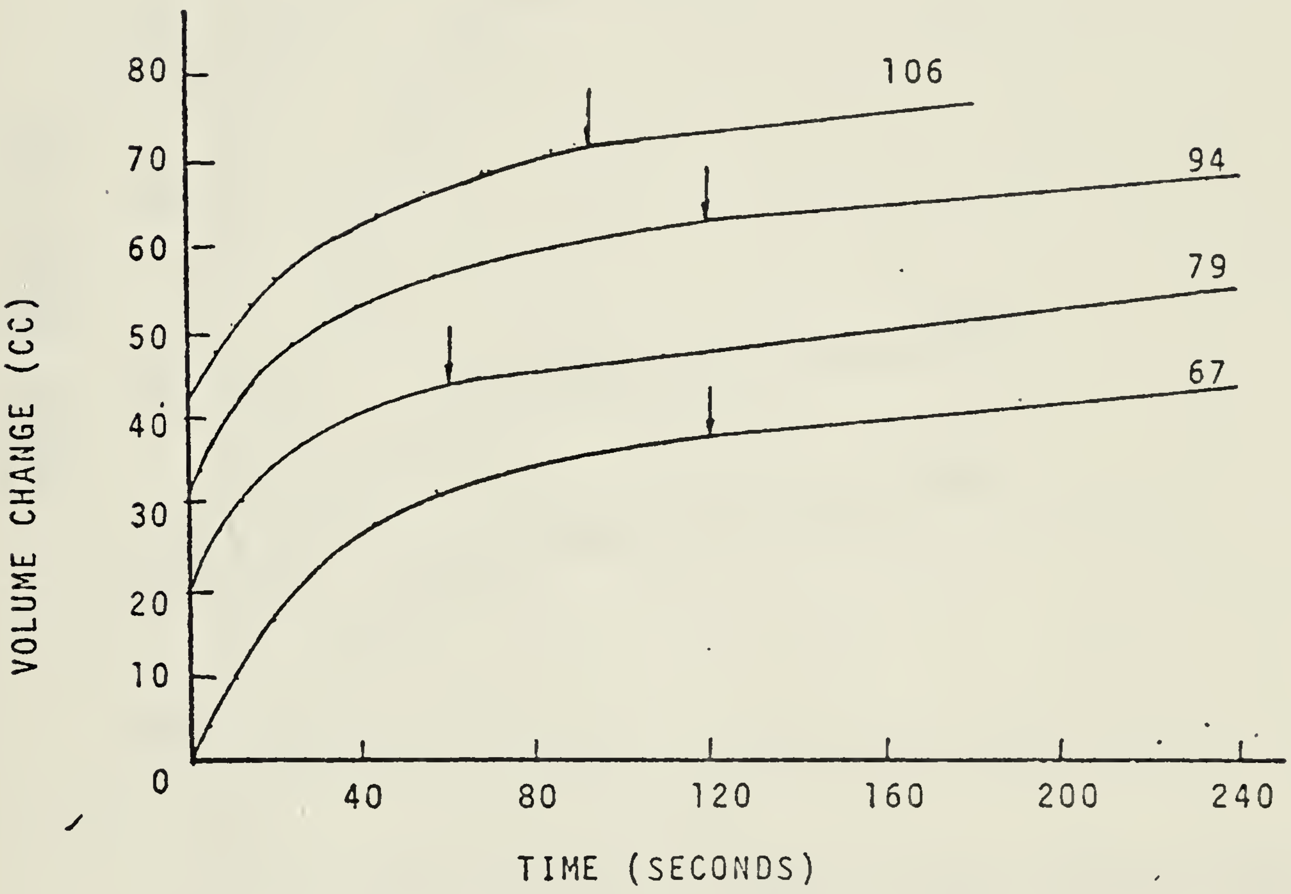

VOLUME CHANGE (CC)
250
200
150
100
50
0
40
60
80
100
120
PRESSURE (PSI)

FIGURE B9

FIELD TEST RESULTS
MOUNT BLACKSTRAP

DEPTH: 70'
MATERIAL: TILL (FLORAL)
MODULUS: 7000 PSI



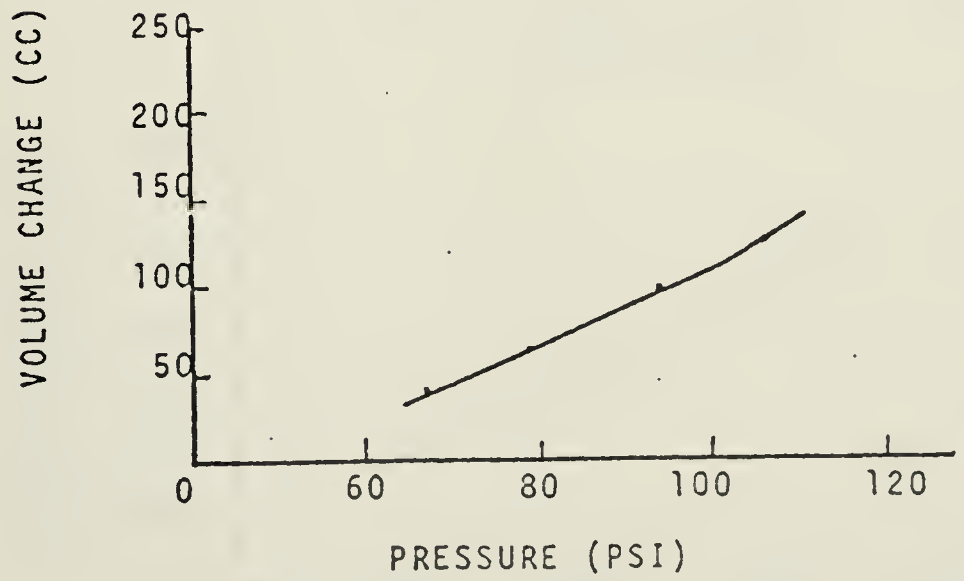
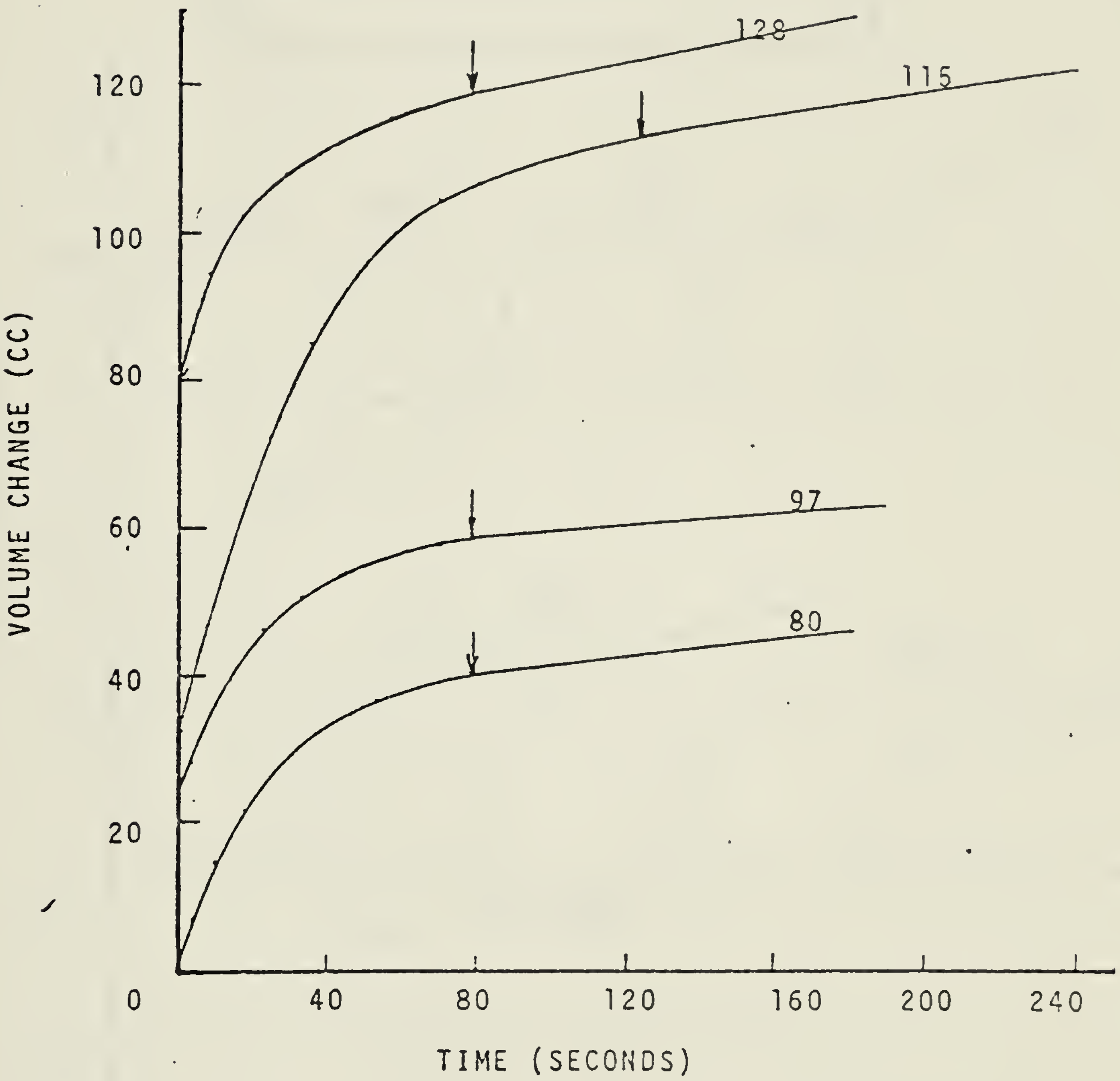


FIGURE B10

FIELD TEST RESULTS
MOUNT BLACKSTRAP

DEPTH: 89'
MATERIAL: TILL (FLORAL)
MODULUS: 6700 PSI



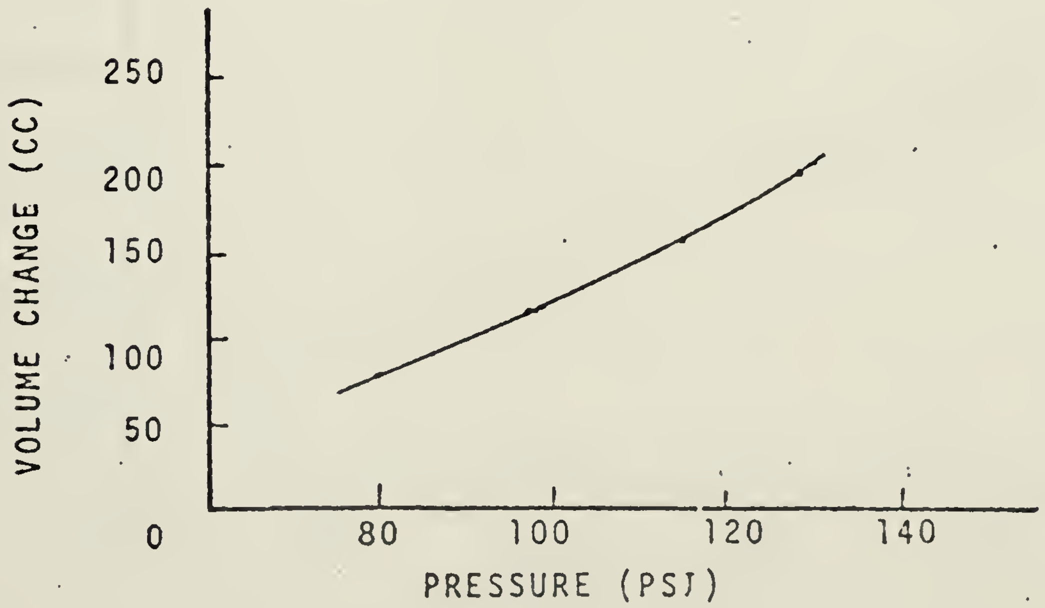


FIGURE B11

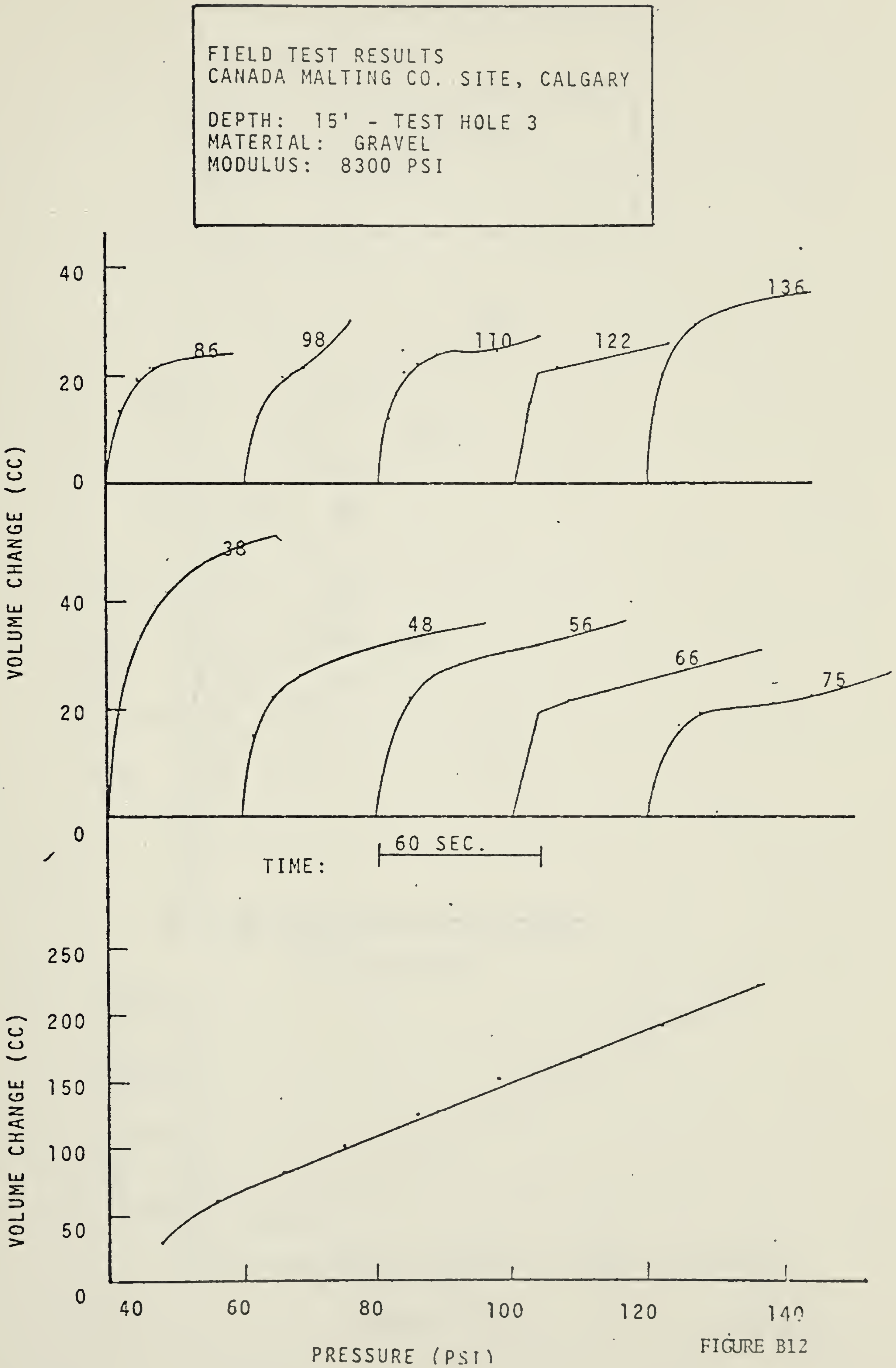

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY
DEPTH: 15' - TEST HOLE 3
MATERIAL: GRAVEL
MODULUS: 8300 PSI
40
20
0
86
98
110
122
136
38
48
56
66
75
VOLUME CHANGE (CC)
TIME:
60 SEC.
250
200
150
100
50
0
40
60
80
100
120
140
VOLUME CHANGE (CC)
PRESSURE (PSI)

FIGURE B12

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY

DEPTH: 16' - TEST HOLE 2
MATERIAL: GRAVEL
MODULUS: 5500 psi

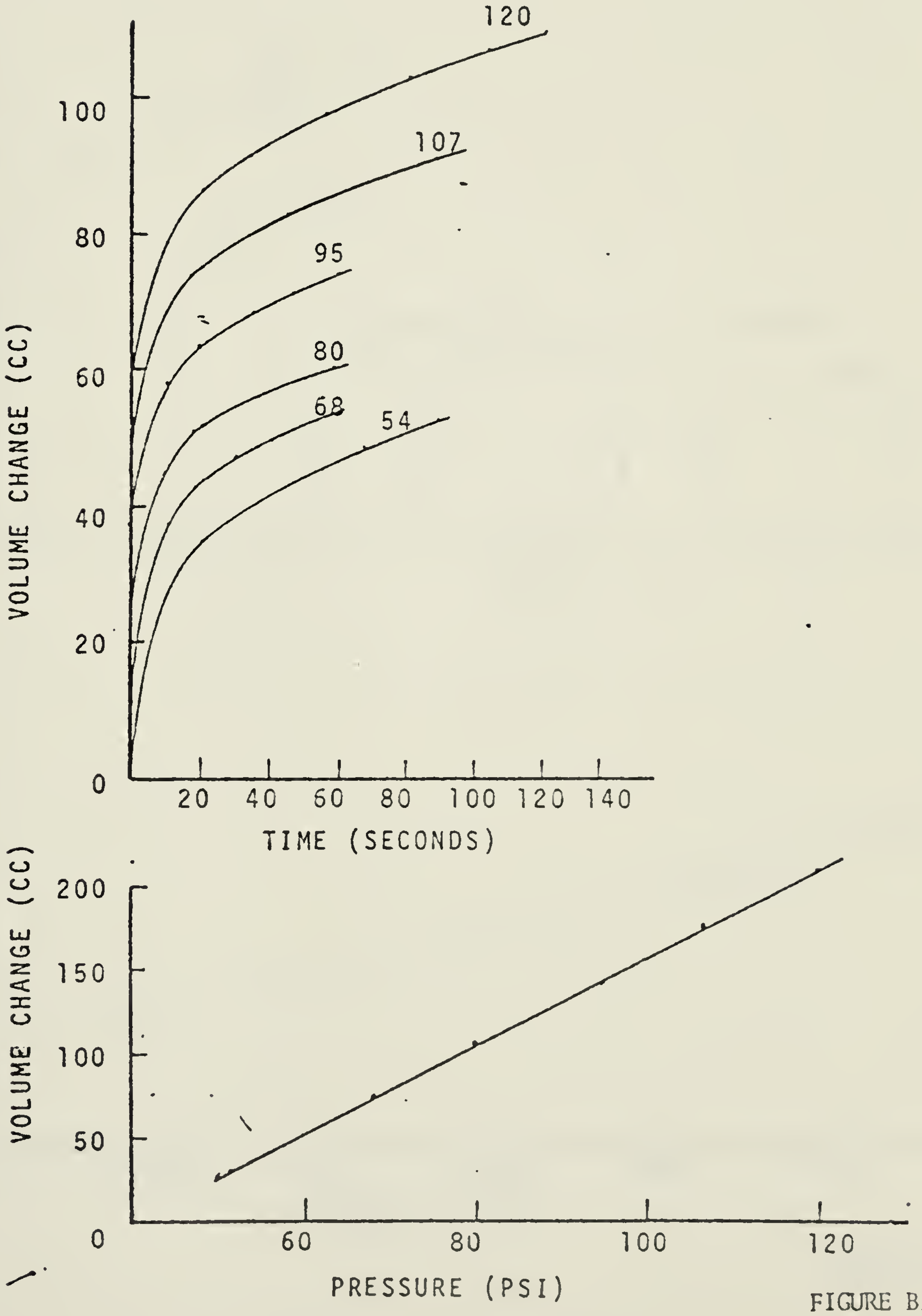
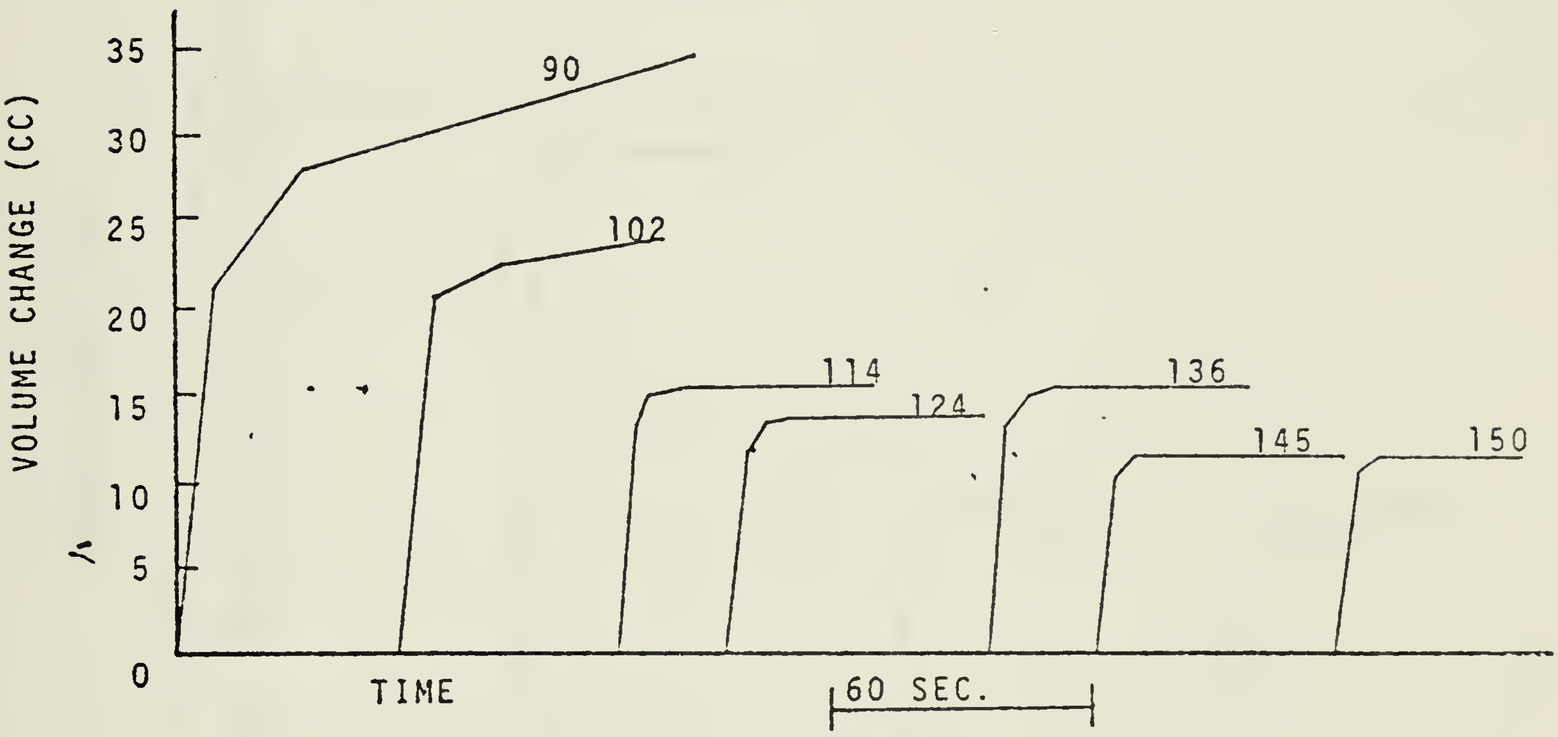


FIGURE B13

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY

DEPTH: 40' - TEST HOLE 1
MATERIAL: BEDROCK (WEATHERED SHALE)
MODULUS: 9800 PSI



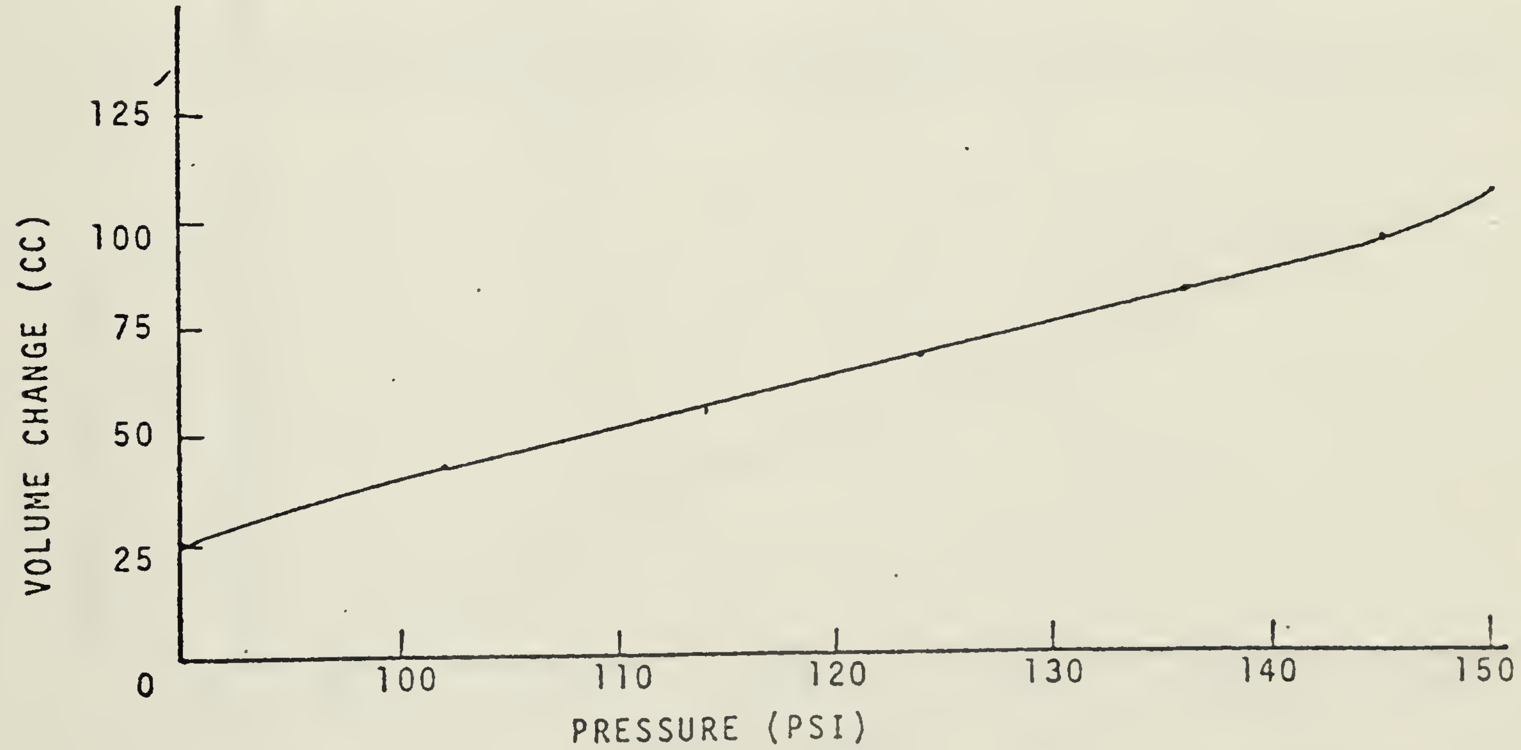
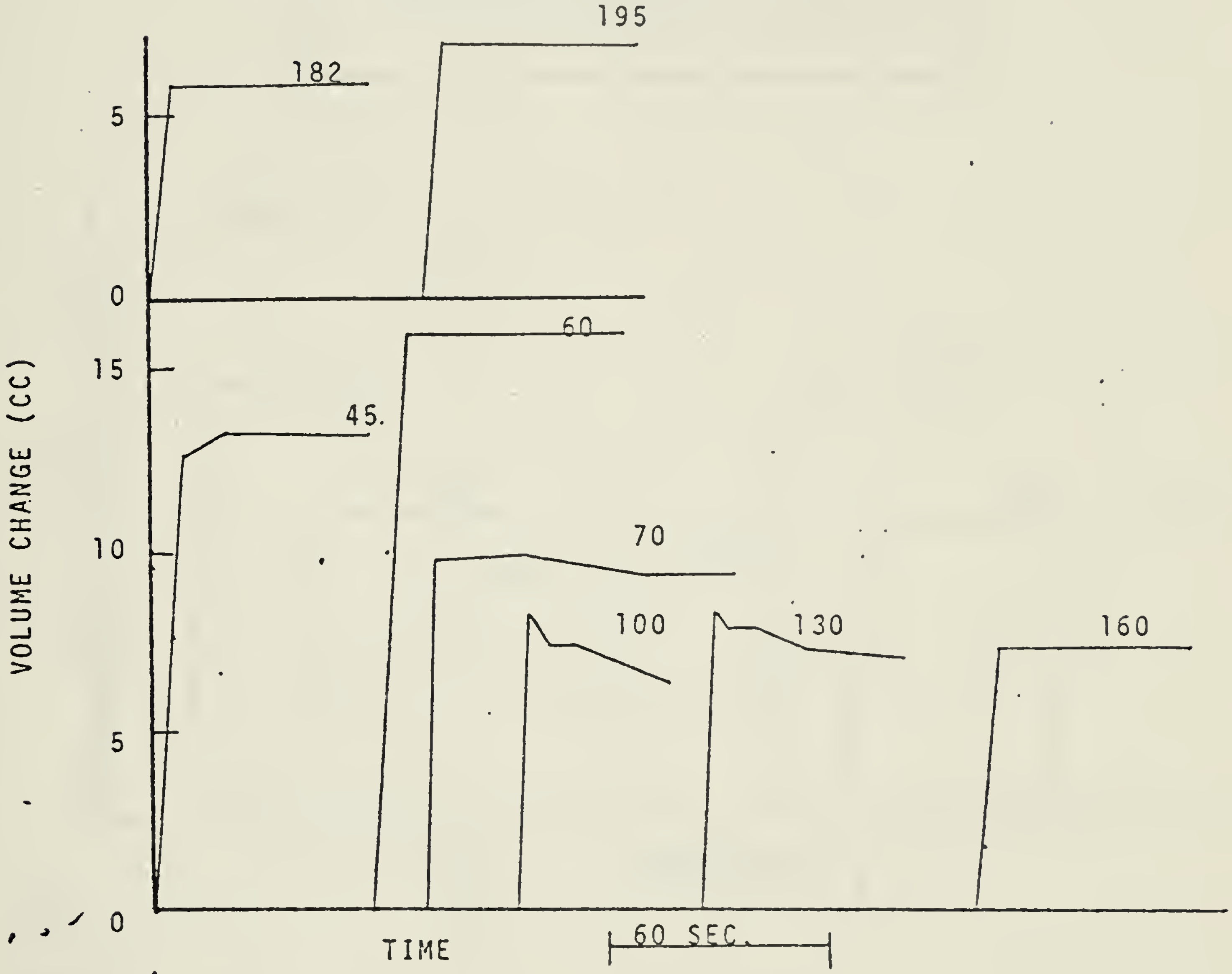


FIGURE B14

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY

DEPTH: 45'
MATERIAL: SHALE
MODULUS: 23,400 PSI



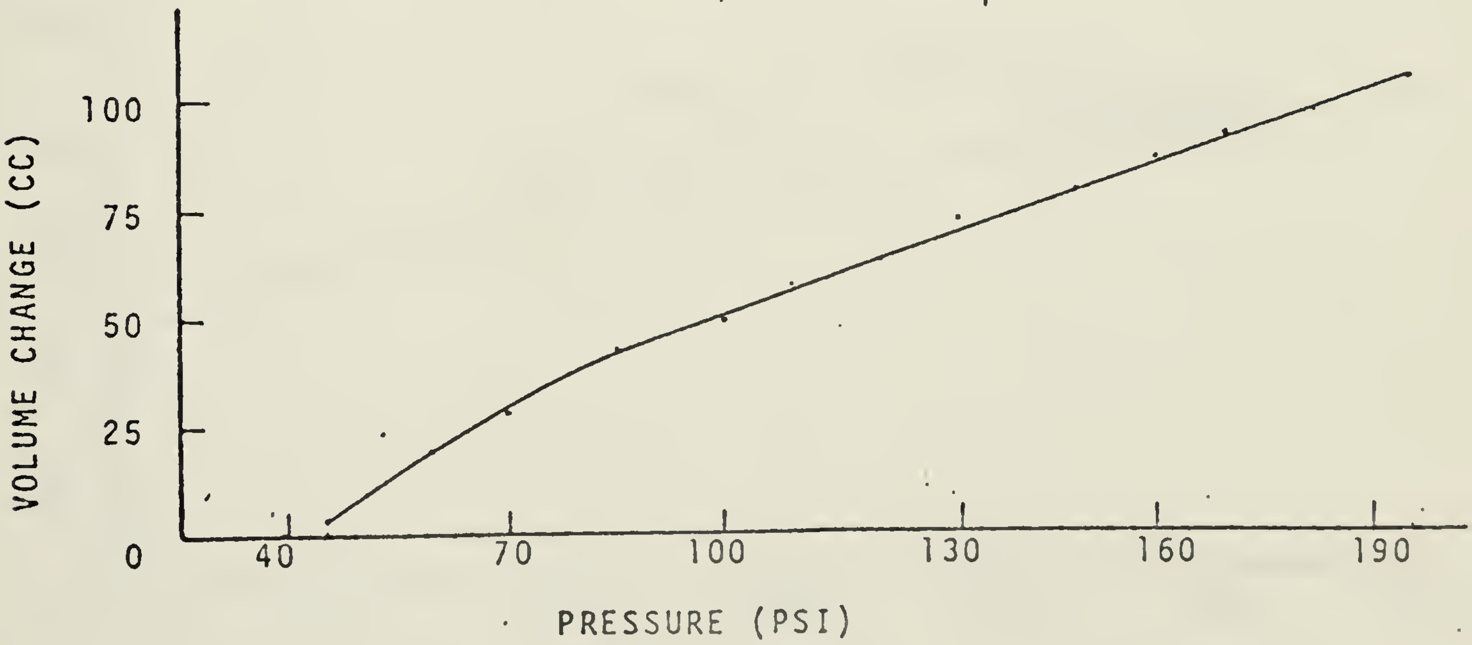


FIGURE B15

FIELD TEST RESULTS
CANADA MALTING CO. LTD., CALGARY

DEPTH: 50'
MATERIAL: SHALE
MODULUS: INITIAL: 17,500 PSI
SECONDARY: 41,500 PSI

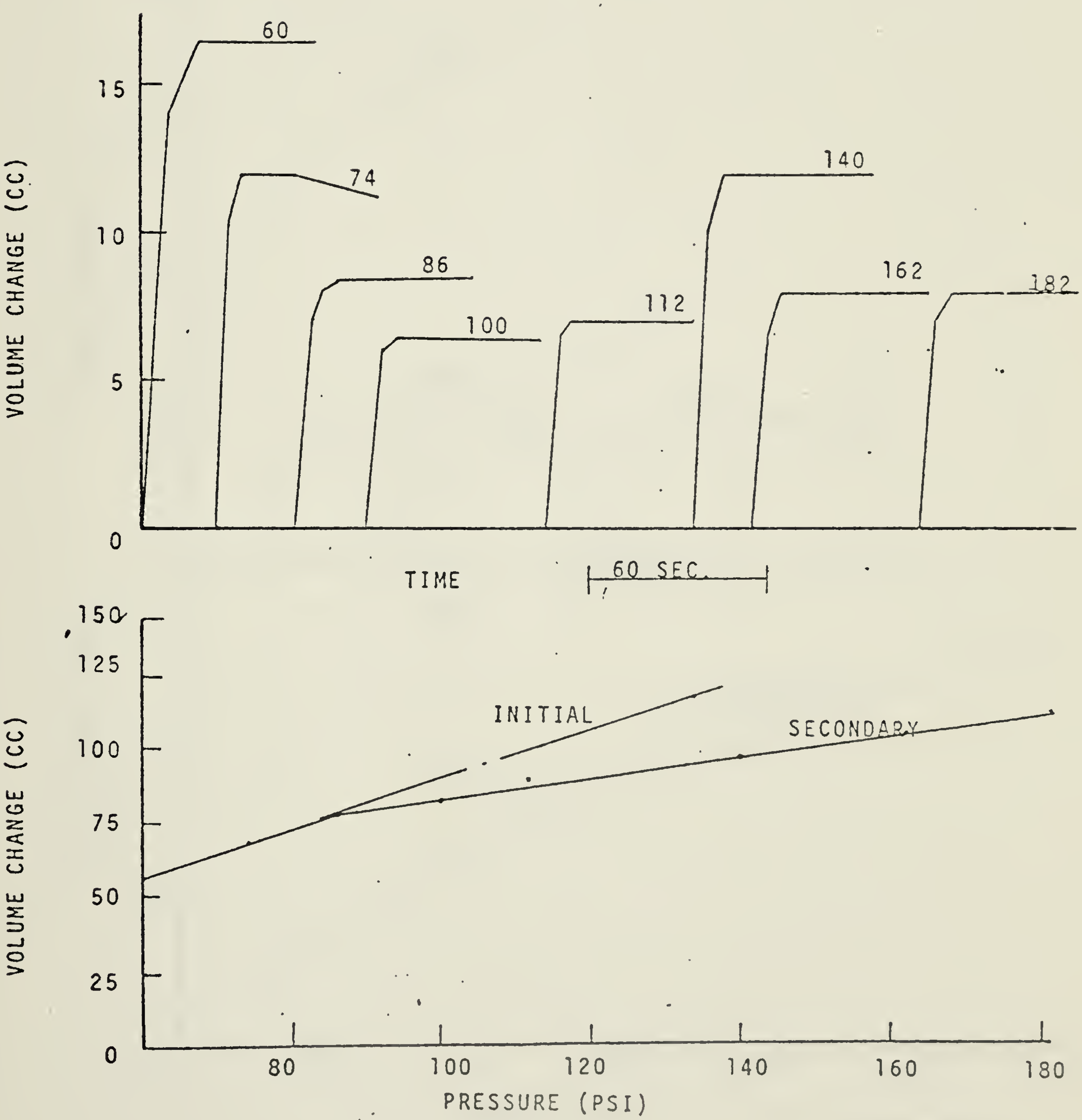


FIGURE B16.

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY

DEPTH: 60'
MATERIAL: SHALE
MODULUS: INITIAL 15,900 PSI
SECONDARY 21,300 PSI

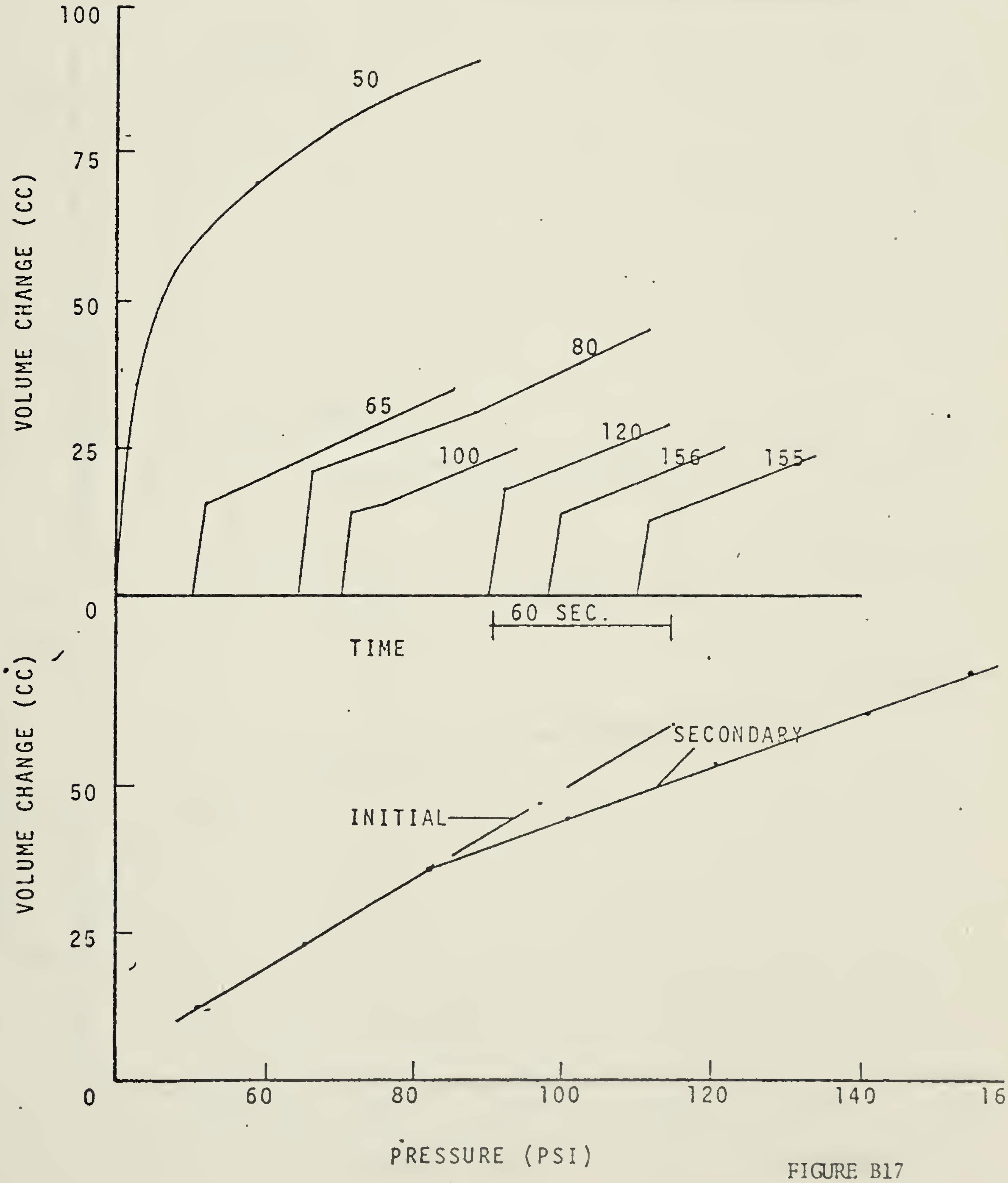


FIGURE B17

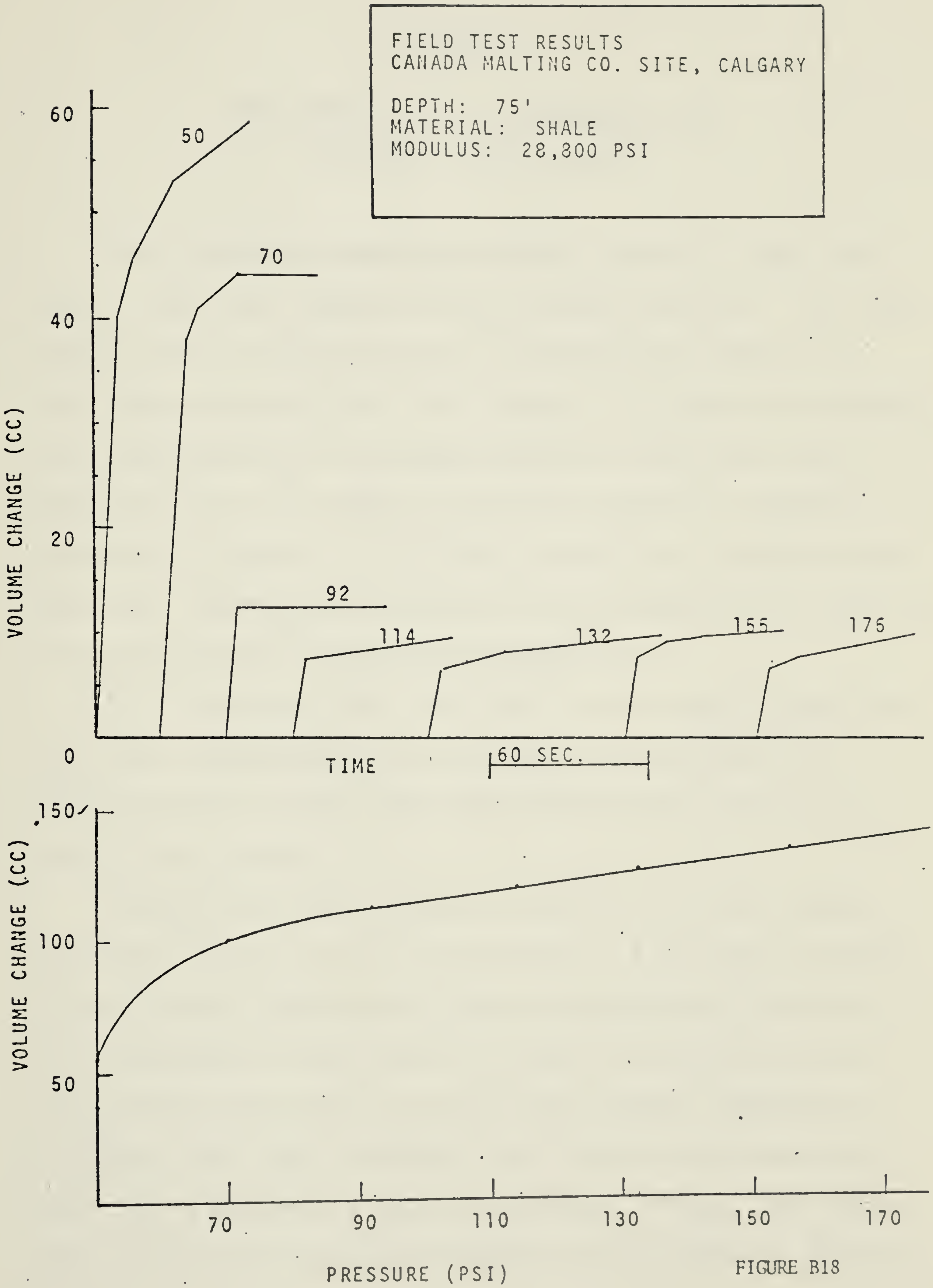

FIELD TEST RESULTS
CANADA MALTING CO. SITE, CALGARY
DEPTH: 75'
MATERIAL: SHALE
MODULUS: 28,800 PSI
VOLUME CHANGE (CC)
60
40
20
0
50
70
92
114
132
155
176
TIME
60 SEC.
VOLUME CHANGE (CC)
150
100
50
70
90
110
130
150
170
PRESSURE (PSI)

FIGURE B18

APPENDIX C

RECOMMENDED FIELD PROCEDURES AND OPERATION OF THE PROBE

The instrument developed in the course of this study has a 4 5/8 inch diameter at the largest section. The probe can be used in the definition of deformation moduli for test holes varying from 4 3/4 inches to 6 inches in diameter. The determination of strength parameters from the probe requires that it be used in a borehole having a maximum diameter of roughly 5 5/8 inches, which is a standard rock-bit size. Procedures outlined in this section will refer to use of the probe for modulus determination.

In its present form, the U of A probe must be used in test holes containing drilling mud. The addition of a solenoid valve on the probe would facilitate use of the probe in dry holes.

The most suitable borehole diameter is 5 5/8 inches. The probe has been used in boreholes of 4 3/4 inch diameter but not without difficulty during insertion and retrieval. An air pressure of the order of 10 psi is required in the probe during lowering to provide the external membrane of the probe with some stiffness such that an all-round pressure can be applied by the drilling mud. Under zero inflation pressure the external membrane on the probe is able to

collapse sufficiently to eliminate the effect of the hydrostatic stress imposed by the drilling mud in the hole. Under these conditions an all-round pressure roughly equal to the drilling mud head to the instrument will not be produced. As mentioned earlier, this drilling fluid pressure, as transferred to the internal air pressure of the probe and consequently to the exterior of the measure cell is required to counter the development of the piezometric head on the measure cell during lowering. The specific gravity of bentonitic drilling muds exceeds 1.0. Under an initial air pressure of about 10 psi at the commencement of lowering of the probe, pressures in excess of 15 psia on the water control valve at the control panel can be maintained, eliminating the occurrence of cavitation in the water supply line below this valve.

The use of a drill collar or stabilizer bar is recommended for drilling of test holes of 5 5/8 inch diameter or smaller. This attachment above the drill bit aids in the production of a borehole having no abrupt directional or alignment changes that can inhibit passage of the probe.

Rotary-drilled boreholes are the only type that can be used with confidence in tests for modulus determination. Percussion or shear-displacement drilling methods utilized by such rigs as the churn drill (cable tool) or diesel hammer-driven tools are considered to produce excessive borehole disturbance.

Test drilling should be made at rates compatible with that at which pressuremeter tests can be performed. For the

case of easily softened materials such as clay-shale and siltstone, a thick mixture of drilling mud should be used (consistency such that water pump intake screen openings need continuous scraping). Drilling should be commenced only when performance of a pressuremeter test is assured. Should drilling be advanced to depths greater than can be tested in a period of several hours, re-drilling of the hole may be required, especially when technical problems can develop and inhibit the rate at which tests can be conducted.

For the case of testing under freezing temperatures, replacement of water as a measuring fluid with pure methanol is advised. Mixtures of ethylene glycol or methanol with water have been found to have viscosities that produce prohibitive flow rates in the probe.

<u>OPERATION OF THE U OF A PROBE</u>

The following procedures apply to operations of the probe to depths not exceeding 25 feet in dry test holes or to lower levels in mud-filled test holes.

I. De-Airing the Measure Cell:

Fill the measure cell under an applied water pressure of about 5 psi. Open the bleeding screw on the water return line and allow flow into and out of the measure cell under 5 psi applied pressure. At the same time, continuously squeeze the measure cell portion of the probe until no air bubbles are seen to leave the measure cell. The measure cell will empty fairly

rapidly when being bled. Close the bleed screw and allow cell to fill again. Repeat the above bleeding and air expulsion process several times until the measure cell is de-aired. Allow the measure cell to fill under an applied pressure of 5 - 10 psi in which condition the probe can be readied for lowering or transport. Storage of the probe with the measure cell under internal pressure will obviate de-airing with successive use.

II. Seating the Probe

1. With the measure cell de-aired, filled and under a pressure of about 10 psi the probe is ready for lowering.

2. Fill the reservoir and note the volume reading.

3. Measure the outside diameter of the probe at the midpoint of the measure cell, recording it as a reference diamter.

4. With water supply to the probe turned off, apply approximately 10 psi air pressure to the probe.

5. Lower the probe to the desired test elevation. Observe the air pressure gauge which should increase with increasing all-round pressure on the probe applied by mud in the test hole. Occasionally open the water supply valve to the probe. Water should return from the probe at all times, ensuring that pressures greater than atmospheric

are acting on the control valve and that cavitation in the supply line to the probe will not occur.

6. The adjustable relief valve at the control panel is used to maintain an air-water pressure differential of 10 psi. The valve is used on the air or water phase at the panel such that the piezometric head to the measure cell plus the applied pressure on the water phase are at all times only 10 psi greater than applied air pressure at the probe. Up to piezometric heads of 23 feet, measured from center of measure cell to midpoint of reservoir, the air or guard cell pressure will have to be reduced at the control panel, 23 feet corresponding to a piezometric head of 10 psi. Beyond heads of 23 feet, pressure applied to the water phase at the panel will have to be reduced by the relief valve. For example, if the piezometric head to the measure cell is 50 psi, the relief valve at the control panel will be between the pressure cylinder and the water reservoir and will be set to a cracking pressure of 40 psi. In this way, no pressure will be applied to the reservoir until the applied air pressure at the probe is 40 psi and the following conditions will be met:

Piezometric head at probe: 50 psi

Adjustable relief valve cracking pressure: 40 psi

i) applied guard cell pressure = 40 psi

applied water pressure = 0

piezometric pressure = 50 psi

water pressure on measure cell = 50 psi

required differential = 10 psi

ii) applied guard cell pressure = 160 psi

applied water pressure = 120 psi

piezometric pressure = 50 psi

water pressure on measure cell = 170 psi

required differential = 10 psi

In summary:

i) for piezometric heads on measure cell up to 23 feet, G.D. = 10-P, $A < W$

ii) for piezometric heads on measure cell greater than 23 feet, G.D. = P-10, $A > W$

where G.D. = difference between air and water guage pressures as recorded on the control panel guages. The guage difference required to obtain the required 10 psi air-water differential at the probe is equal to the cracking pressure of the adjustable relief valve on the control panel.

P = piezometric head in pounds per square inch, measured from midpoint of measure cell to mid-point of water reservoir at control panel

A = applied air pressure (guard cell pressure) as recorded on air pressure gauge at panel.

W = applied water pressure as recorded on water pressure guage off reservoir at panel.

7. With the adjustable relief valve set to produce the required air-water differential, air pressure roughly equal to estimated total horizontal ground stresses should be applied, flow allowed to the measure cell and volume change readings observed. The volume change can be recorded on combined flow from sight glass and reservoir. When volume change with time is essentially zero, the probe is seated.

8. Turn off water to the probe, record the sight glass reading and if necessary refill the reservoir.

9. Isolate the reservoir from volume change measurement, using only the sight glass to record flow to the probe.

10. Apply a pressure increment of 15 psi or so to the air phase. After 10 seconds have elapsed, open the flow control valve to the measure cell and record sight glass readings at 10 second intervals for 60 seconds, 30 second intervals from then on.

11. Volume change will be essentially linear with time after 60 seconds or less. Continue readings for another 2 or 3 minutes (maximum) and then turn off flow to measure cell, record reading and readjust sight glass level if necessary.

12. At some convenient time (30 to 60 seconds)

after turning off flow to probe and making any required adjustments, apply the next pressure increment, wait 10 seconds and turn on flow to measure cell. Record as noted above.

13. After 4 or 5 pressure increments in excess of estimated lateral ground stress, shut off gas supply from the pressure source and slowly reduce reservoir pressure until fluid returns from the probe. Maintain this reservoir pressure until fluid volume in the reservoir and sight glass has been restored to original quantities prior to seating the probe.

14. Deflate the probe to approximately 10 psi. Do <u>not</u> attempt to raise or lower the probe under applied air pressures in excess of 10 psi.

APPENDIX D

TECHNICAL DOCUMENTATION OF THE U OF A PRESSUREMETER

Figures D1, D2 and D3 constitute the technical documentation of the equipment.

In schematic form, the probe and control system are described.

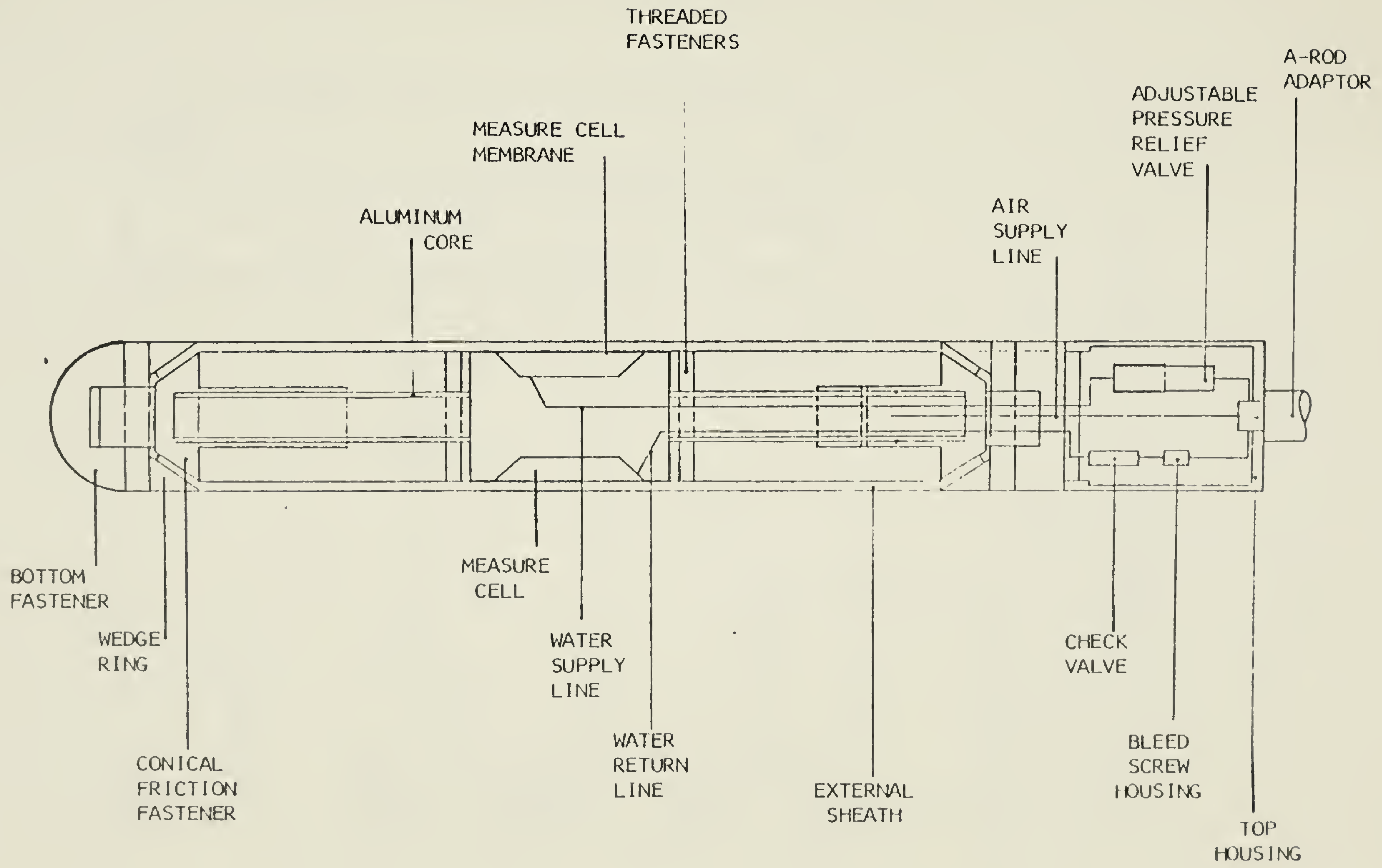

THREADED FASTENERS
A-ROD ADAPTOR
ADJUSTABLE PRESSURE RELIEF VALVE
MEASURE CELL MEMBRANE
AIR SUPPLY LINE
ALUMINUM CORE
BOTTOM FASTENER
MEASURE CELL
WEDGE RING
WATER SUPPLY LINE
CHECK VALVE
CONICAL FRICTION FASTENER
WATER RETURN LINE
BLEED SCREW HOUSING
EXTERNAL SHEATH
TOP HOUSING

FIGURE D1

SCHEMATIC OF CONTROL PANEL

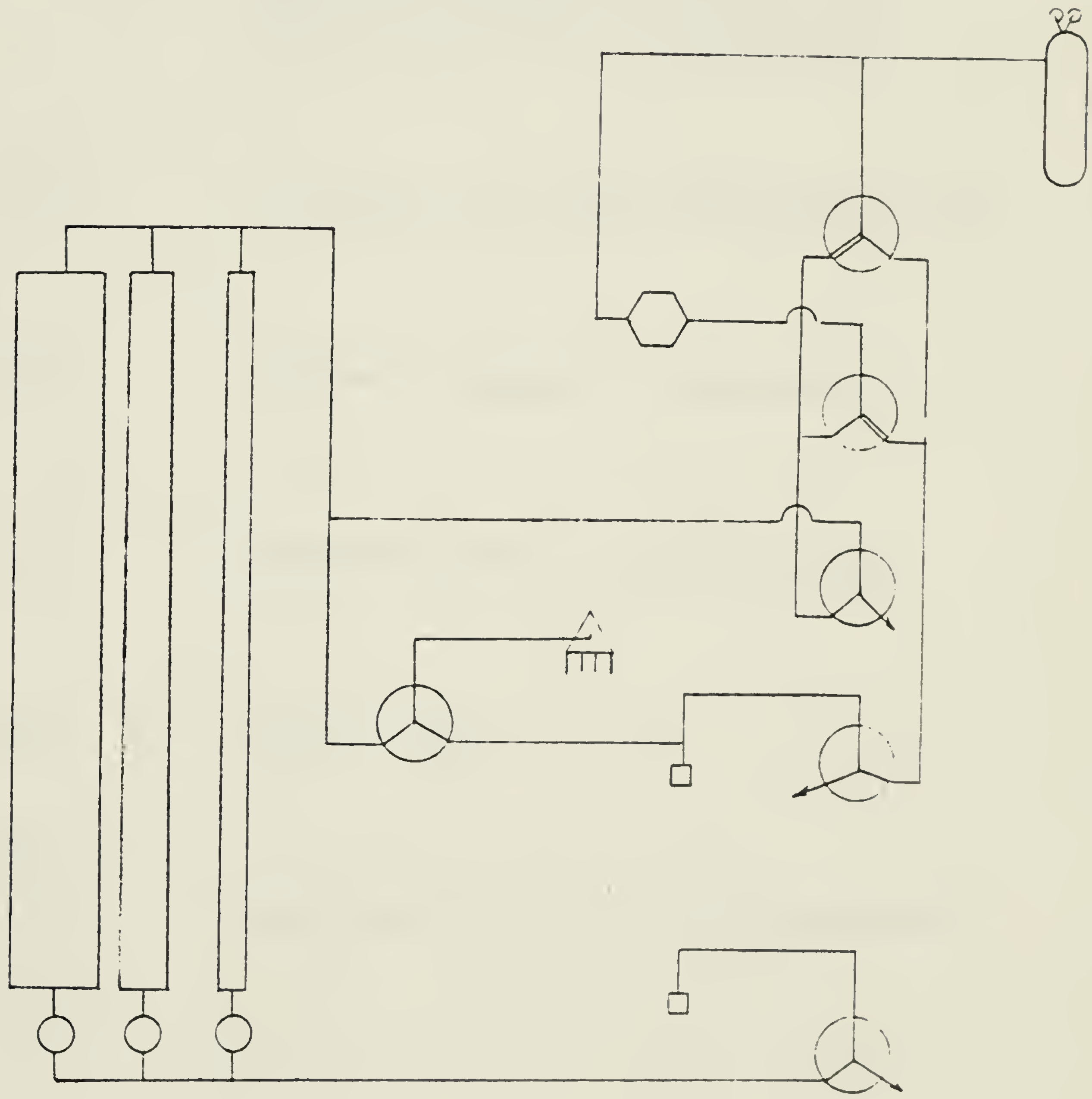
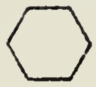


FIGURE D2

EXPLANATION OF SYMBOLS

3 Way Valve (common at top)
Double line indicates routing or "on" position for full pressure to reservoir, reduced pressure to probe air. Opposite position is for full pressure to probe air, reduced pressure to reservoir.



Air Regulator (fine adjustment type)

Pressure Sensor or Transducer

Quick-Connect to External Lines

On-Off Valve

Compressed Air Source and Regulator

FIGURE D3

B30140